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METCALF AND EDDY

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SEWERAGE AND SEWAGE DISPOSAL

A TEXTBOOK

SECOND EDITION

783 pages, 6 × 9, 224 illus., 111 tables

SEWERAGE
AND
SEWAGE DISPOSAL

A TEXTBOOK

BY

LEONARD METCALF

AND

HARRISON P. EDDY

SECOND EDITION
REVISED BY HARRISON P. EDDY
TENTH IMPRESSION

McGRAW-HILL BOOK COMPANY, INC.
NEW YORK AND LONDON
1930

PRINTED IN THE UNITED STATES OF AMERICA

THE MAPLE PRESS COMPANY, YORK, PA.

PREFACE TO THE SECOND EDITION

This book has been almost entirely re-written, with the object of making it more valuable as a text book for class use, and of bringing it up to date in its discussion of principles and practices. To make it better suited for teaching, the assistance of Professor Richard G. Tyler of the Massachusetts Institute of Technology[1] and of Professor Gordon M. Fair of Harvard University was enlisted: and they have had much to do with putting the book in the shape in which it now appears.

The basis for this book, as was the case with the first edition, is the authors' three volume treatise on American Sewerage Practice. The recent revision of Volume I of that work provided the greater part of the new material for the portion of this book relating to design of sewers. Other new material has been taken from various sources, most of which are referred to in the text or listed in bibliographies.

Before undertaking the revision, letters were sent to all the professors who were known to have used the book in their classes, with a request for suggestions as to the manner of making the book more useful. Many helpful comments were received. It is hoped that the criticisms of the first edition have been met in this one.

That the arrangement of material, especially in the portion of the book devoted to disposal of sewage, has been considerably changed, will be obvious to anyone familiar with the first edition. It is believed that the new arrangement is more logical and better adapted to class work.

The former chapter on "Cost Estimating" has been replaced by one on "Financial Considerations." The subject of cost estimating is treated more briefly, but it may be that the student will obtain from it quite as accurate a conception of the principles involved and of their proper application as from the former more lengthy discussion. More than this appears unnecessary. There has been added a short discussion of the methods of financing the construction and operation of sewerage works—by bonding, by general taxation, by special assessments, and by

[1] Now Dean of Engineering, University of Washington.

service charges. The economic considerations relating to these methods are discussed and explained, although not in great detail, the object being to provide the student with a sound understanding of the basic principles which apply to any case.

The death of Mr. Metcalf, in January 1926, left upon the surviving author the responsibility for this revised edition. He gratefully acknowledges his indebtedness to Professors Tyler and Fair, whose assistance has already been noted; to his partner Charles W. Sherman, who has edited the entire book for printing; and to other partners and various members of the staff of Metcalf and Eddy who have aided in its preparation, more particularly John W. Raymond, Jr., George C. Houser and Philip W. Taylor.

HARRISON P. EDDY.

BOSTON, MASS.
December, 1929.

PREFACE TO THE FIRST EDITION[1]

Since the three volumes of "American Sewerage Practice" were published in 1914–1915, the authors have been urged to prepare a single-volume abridgment of them for class use in engineering schools. The authors were reluctant to do this, because of two conditions. The first is the great variation in the time devoted to sewerage engineering in different schools. This has the result that a textbook so brief as to verge on the superficial from the viewpoint of one class may be too detailed for another. The second condition is the fact that some of those features of sewerage engineering which in practice it is most important to master are apparently held by those laying out college courses to deserve little attention from students. For example, a wise estimate of the quantities of sewage and storm water to be handled is the basis of design of a sewerage system, yet the methods of preparing such estimates and actual practice in their use receive relatively little attention in college courses. Furthermore, the best arrangement of the main sewers in a sewerage plan may depend largely upon the treatment, if any, that the sewage must receive before disposal and upon the place of disposal, subjects involving a knowledge of the nature of sewage and the changes which it undergoes under natural conditions or those involved in specific methods of treatment. Yet some college classes in sewerage engineering receive no instruction in sewage treatment.

As the requests for a textbook continued, the authors finally decided to prepare a book giving the information which they consider it desirable for the young student to acquire before taking up work in this field. In a sense, they have felt under obligations as professional engineering specialists to do their part in smoothing the way for the first steps taken by engineering students in this branch of engineering.

This book can be used in two ways. The text can be read and mastered within the time allotted to the subject even when the course is very short. Where more time is available, the

[1] Portions of the original preface have been omitted in this revision.

authors believe that the student will gain a much better grasp of the fundamentals of the subject by devoting most of his additional time to the solution of problems. He should also lose no opportunity to see sewerage construction in progress and sewage treatment works in operation.

There is little in this textbook not covered in much greater detail in "American Sewerage Practice." The three volumes of that treatise have elaborate indices, so that it seemed unnecessary to insert references to the treatise in this textbook. The student is assumed to have access to textbooks on hydraulics, masonry and reinforced concrete. No cost data are given, because the great changes in costs induced by the war make it necessary to employ older data with a great deal of caution. They must be supplemented, too, by practical knowledge of current changes which very few undergraduate students possess.

The authors are engineers, not teachers, hence this text book reflects the engineer's rather than the teacher's viewpoint. They will appreciate the more, therefore, any suggestions for increasing its usefulness to undergraduate students and their instructors. Constructive criticism is invited, because through it the experience of the authors and other engineers in designing, building and operating sewerage and sewage treatment works, can be made more helpful to the students who will eventually carry the responsibility for such undertakings.

The authors take pleasure in giving recognition to Mr. John M. Goodell, for many years Editor of "Engineering Record," who assisted them in the preparation of "American Sewerage Practice" and who has done the greater part of the work of condensing its three volumes into this book. Credit is also due to the junior partners and members of the staff of Metcalf and Eddy, and to the publishers, for their helpful assistance.

As the book reflects the past experiences and current opinions of many practicing engineers, it should be of service to the student.

LEONARD METCALF.
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14 BEACON STREET,
BOSTON, MASS.
December, 1921.

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SEWERAGE AND SEWAGE DISPOSAL
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CHAPTER I

INTRODUCTION

1. Wastes.—The wastes produced by a municipality consist of *sewage* and *municipal refuse;* the former is liquid, made up of the used water supply of the community, containing bodily discharges or excreta, household and industrial wastes; the latter comprises the solid wastes,—*garbage, rubbish, ashes, street sweepings, stable refuse, offal or market refuse, dead animals,* and *industrial refuse* (including waste building materials). The prompt, continuous and sanitary removal of wastes to suitable places for disposal, usually situated at some distance from residential districts, is essential to the health and convenience of the community.

Sewage has been defined by the Committee on Sewerage and Sewage Disposal of the American Public Health Association as "a combination of (*a*) the liquid wastes conducted away from residences, business buildings and institutions and (*b*) from industrial establishments, with (*c*) such ground, surface and storm water as may be admitted to or finds its way into the sewers." It is extremely putrescible, its decomposition produces large quantities of malodorous gases, and it may contain numerous pathogenic or disease producing bacteria such as dwell normally in the intestinal tract or are present in certain industrial wastes, as from tanneries or laundries. Its immediate and unobtrusive removal from its source in residences and industrial plants, to a point sufficiently remote from densely settled areas to permit of its discharge or suitable treatment without offense, is most desirable.

2. Sewers.—The removal of sewage is usually accomplished through underground conduits called *sewers,* though excreta

are sometimes collected and removed separately in pails or other containers. Where the sewers carry the household and industrial wastes only, they are called *separate sewers.* Those carrying storm water from the roof and street surfaces are known as *storm-water drains,* while those carrying both sewage and storm water are called *combined sewers.* The *building connection* (sometimes called *house sewer*) connects the house *plumbing* or drainage system to the *lateral sewer* in the street. The *laterals* discharge into *sub-main* or *main* sewers, which in turn discharge through *outfall* and *trunk sewers* or *intercepters* to the point of disposal. *Intercepters* are generally laid transversely to the general sewer system to intercept the dry-weather flow of sewage and such additional surface and storm water as may be desirable. *Relief sewers* are designed to carry a portion of the flow from a district already provided with sewers of insufficient capacity, and thus prevent over-taxing the latter, while *storm-water overflow sewers* are designed to carry the excess over a certain volume of combined sewage from a main or intercepting sewer to an independent ontlet.

HISTORICAL

3. Sewers of Antiquity.—The first record of a sewer which Curt Merckel, the antiquarian of engineering, has been able to find is on an old Babylonian seal cylinder. Layard's explorations revealed arched sewers in Nineveh and Babylon dating from the Seventh Century before Christ. Schick and Warren have unearthed considerable information about the sewers of Jerusalem; the works of this class in Grecian cities are fairly well known, and the great underground drains of Rome have repeatedly been described. It is known, however, that these channels and conduits were not used to any substantial extent by means of direct connections to them from the houses, for the requirements of public health were little recognized then and compulsory sanitation would have been considered an invasion of the rights of the individual. Livy states that the Roman building regulations only stipulated that the house connections were to be made at the cost of the property owners. Public latrines were doubtless used by most of the people and it is probable that the gutters were the chief receptacle of the ordure of the city, which was washed thence into the sewers. These must have been extremely offensive when not flushed, for other-

wise the regular delivery of water for the purpose of cleaning them would not have been so emphasized in the following notes by Frontinus, a water commissioner of the city whose valuable notes of engineering work have been edited by Clemens Herschel:

> I desire that nobody shall conduct away any excess water without having received my permission or that of my representatives, for it is necessary that a part of the supply flowing from the water castles shall be utilized not only for cleaning our city but also for flushing the sewers.

4. The Beginning of Modern Sewerage Practice.—It is astonishing that from the day of Frontinus (the First Century, A.D.) to that of W. Lindley,[1] there was no marked progress in sewerage. The renaissance began in Hamburg, where a severe conflagration destroyed part of the city in 1842. The portion ruined was the oldest section and it was decided to rebuild it according to modern ideas of convenience. This work was intrusted to Lindley, who carried it out in a way that aroused warm praise among engineers of a somewhat later period, when the test of service had placed the seal of approval on his plans. Twenty-five years after the sewers were completed they were found by a committee of experts to be clean and almost without odor.

The sewerage of Hamburg, while indicative of an awakened public recognition of the need of improvement in such works, was hardly the result of any real appreciation of the value of sanitation but was, rather, the result of business shrewdness in taking advantage of exceptional local conditions to plan streets and sewers to answer in the best way the recognized needs of the community and the topographical conditions. The history of the progress of sanitation in London probably affords a more typical picture of what took place quite generally about the middle of the Nineteenth Century in the largest cities of Great Britain and the United States.

5. Early London Sewerage.—A statute was passed in 1531 in Henry VIII's reign[2] and amended in that of William and Mary which afforded the legal basis of all sanitary works of sewerage well into the Nineteenth Century. For a period of about three hundred years, while London outgrew the narrow

[1] Lindley was one of the leading English engineers of his day, Sir Robert Rawlinson being his only rival at the head of the sanitary branch of the profession. He became thoroughly identified with German work, however, first at Hamburg and later at Frankfort.

[2] Statutes relating to local drainage problems had been passed in the reigns of Henry III, Henry VI, and Henry VII.

limits of the city proper and its adjacent parishes and became a great metropolis, the center of the world's commerce, sanitation was as little considered as magnetism or the utilization of steam for power purposes. The city was better off than most of the metropolitan district, for it had Commissioners of Sewers elected annually by the Common Council from its members. They had power over all conditions relating to public health and comfort and had authority to appoint a medical officer of health. But the city was only a small part of the metropolitan area, 720 out of 75,000 acres in 1855, with only 128,000 out of a total population of 2,500,000 and less than 15,000 out of a total of 300,000 houses. Outside the city, the methods of local government were chaotic; in some of the parishes, surveyors of highways were appointed to do very restricted engineering work, and in eight there were Commissioners of Sewers, apparently having powers modeled after those of London but less extensive.

This lack of central authority rendered a systematic study and execution of sewerage works impossible. As late as 1845 there was no survey of the metropolis adequate as a basis for planning sewers. The sewers in adjoining parishes were at different elevations so that junctions of them were impracticable. Some of the sewers were higher than the cesspools which they were supposed to drain, while others had been so constructed that for them to be of any use the sewage would have had to flow uphill. Large sewers were made to discharge into small sewers.[1]

The first engineer to make a comprehensive study of metropolitan sewerage needs in an official capacity, John Phillips, gave this testimony of the condition of London basements and cellars in 1847:

> There are hundreds, I may say thousands, of houses in this metropolis which have no drainage whatever, and the greater part of them have stinking, overflowing cesspools. And there are also hundreds of streets, courts and alleys that have no sewers; and how the drainage and filth are cleaned away and how the miserable inhabitants live in such places, it is hard to tell.
>
> In pursuance of my duties from time to time, I have visited very many places where filth was lying scattered about the rooms, vaults, cellars, areas, and yards, so thick and so deep that it was hardly possible to move for it. I have also seen in such places human beings living and sleeping in sunk rooms with filth from overflowing cesspools exuding through and running down the walls and over the floors . . . The effects of the effluvia, stench, and poisonous gases constantly evolving from these foul accumulations

[1] JEPSON: "Sanitary Evolution of London."

were apparent in the haggard, wan, and swarthy countenances and enfeebled limbs of the poor creatures whom I found residing over and amongst these dens of pollution and wretchedness.

6. Cholera Outbreaks.—In 1847, scared by an outbreak of cholera in India, which had begun to work westward, a royal commission was appointed to inquire into sanitary improvements for London. This body reported that the sewerage of the entire metropolitan district should be handled by a single board, and in 1848 Parliament followed this advice and created the Metropolitan Commission of Sewers. That body and its successors in the office unfortunately failed to measure up to their opportunities; they produced reports showing clearly the need of extensive sewerage works and other sanitary improvements, and built the Victoria sewer at great expense, which fell into ruins not many years later, but did little more. In the summer of 1848 cholera was discovered in London, and before the winter was over it claimed 468 victims. It broke out again in the spring of 1849 and before it ended about 14,600 deaths were recorded, as against 6,729 in London in the 1832–1833 epidemic.

In 1852 cholera again appeared and in 1853 it slowly gained a foothold. In 1854 it ran its terrible course, claiming a mortality of 10,675 in the last half of that year. The connection between a contaminated water supply and the rapid spread of the disease was clearly shown, but it was also apparent that the filthy living conditions in most houses, due to the absence of effective sewerage, was a great hindrance in combatting the scourge. In 1855 Parliament passed an act "for the better local management of the metropolis;" this laid the basis for the sanitation of London and provided for the Metropolitan Board of Works which soon after undertook an adequate sewerage system.

Apparently, it was not until 1852, when J. W. Bazalgette became chief engineer of the Commission appointed in 1848, that any beginning was made in formulating policies, although at least two engineers of high standing connected with the local works of certain subdivisions of the metropolitan district had been making valuable studies. Bazalgette seems to have possessed the executive ability previously lacking; he tentatively developed plans for interception and then worked them out in detail in collaboration with W. Haywood, the unusually gifted and highly respected engineer of the City Commissioners of Sewers, who had a thorough local engineering experience

and was responsible for many of the basic assumptions upon which the plans were prepared. But no action was taken on these plans until the Metropolitan Board of Works appointed Bazalgette as its engineer and he had been compelled to uphold them against lay and engineering criticism for several years. The works were not actually undertaken until 1859.

7. Early Estimates of Sewage Flow.—In designing these great intercepting and outfall sewers to remedy the insanitary conditions, Bazalgette adopted a mean velocity of 2.2 ft. per second as adequate to prevent silting in a main sewer half full, "more especially when the contents have been previously passed through a pumping station." The computation of the house sewage was based on an average density of population of 30,000 persons per square mile except in the outlying districts, where it was assumed at 20,000. The sewage was estimated at the assumed water consumption, 5 cu. ft. per capita daily. One-half of this sewage was assumed to flow off within 6 hours. The storm-water runoff, for which provision was made, was a rainfall at the rate of ½ in. per day received during the 6 hours of maximum sewage flow, with overflows to discharge the excess due to larger amounts through some of the old sewers directly into the river.

It is not surprising, in the light of present information summarized in one of the following chapters, that these estimates proved too low and flooding took place in low-lying districts. As for the minimum mean velocity selected, it was a little higher than that commonly accepted as the minimum by present day engineers, although not as high as is desirable when practicable.

Prior to Haywood and Bazalgette's work on the London intercepting sewers, Phillips and Roe were prominently before the public as sewerage experts, and among English-speaking engineers Roe's Table[1] was used for many years in selecting the size of sewers. It was acknowledged to be entirely empirical and was said to be based on Roe's observations in the Holborn and Finsbury divisions of the London sewers during more than

[1] Roe's Table was not accepted by some contemporary London Engineers, and in 1865, W. Haywood, engineer of the city, who remained for half a century a leading authority on English municipal engineering, stated at a meeting of the Institution of Civil Engineers that there were no reliable gagings of London sewers in existence and that he had never been able to obtain any accurate information regarding such work from either Phillips or Roe. He stated that he had been forced to make extensive gagings in consequence, and these showed that about half the sewage coming daily over the 11 sq. mi. tributary to the gaging stations passed off between 9 A.M. and 5 P.M.

20 years. It gave the areas which could be drained by sewers of various sizes and on various slopes, as indicated by that experience.

It should be said here that sewerage progress elsewhere in England was apparently less opposed than in London. In 1848 Parliament passed a sanitary code applying to all parts of England and Wales except London, and in 1855 it enacted a nuisance removal law for all England; these laws were the basis of the subsequent sanitary progress outside the metropolis for many years. It will be observed, however, that the development of sewerage undertakings in that country was a direct result of the awakening of the people by a succession of epidemics of cholera, for progress did not begin until that disease had twice terrorized the country within a short period.

8. Early Paris Sewers.—The sewerage system of Paris, like that of London, was inaugurated as a result of a cholera epidemic. The system is unique in some ways, although in its early days the Parisian sewers were doubtless little different from the conduits enclosing old brooks or receiving storm water which were constructed in many large cities. The Menilmontant sewer, mentioned in a record of 1412, was of this type, and remained uncovered until about 1750. It intercepted the drainage of the streets on the northern slope of the city's area lying on the right bank of the Seine, and was called the "great drain" (grand égout or égout de ceinture). The part of the city on the left bank of the river was drained by open gutters leading down the centers of the streets to the river.

The first attempt to study the sewerage needs of the city comprehensively apparently was made in 1808, when there were 14½ miles of drains with about 40 outlets into the river, and during the next 24 years about 10½ miles more of drains were constructed. In 1832 the ravages of cholera awakened the authorities to a partial realization of the city's insanitary condition. The following year a topographical survey was made and, with the aid of the maps based upon it, five systems or divisions of sewerage were planned, based on topographical features of the territory rather than on the administrative boundaries which caused so much delay in the development of rational sewerage works at London.

The new sewers built in Paris from 1833 onward were made 6 ft. or more high wherever possible, in the belief that the work-

men employed in cleaning them would discharge their duties more efficiently if they could labor without being forced to take unnatural positions.[1] Toward 1848 the smaller sewers were given a minimum height of 5.5 to 5.9 ft. without exception, and a width of 2.3 to 2.6 ft. at the springing line of the arch, the width at the invert being a trifle less. Humblot says, "These dimensions are too scanty; for getting about easily at least 2 m. height and 1 m. width are needed." When it became necessary later to enlarge some of these small sections to receive water mains, the top was widened out on one side (sometimes on both sides) while the lower part was left narrow, thus producing those sections shaped something like the letter P which have been the subject of strange comments from persons unfamiliar with their origin.

Although there has been a great deal of criticism of the large Parisian sections, it has generally not been taken into account that the sewers of that city were built with a view to removing street refuse as well as sewage and rain water. There are no catchbasins on these great drains, so that everything entering the inlets and not caught in the little baskets suspended in some of them, passes directly into the sewers.

An interesting feature of the work inaugurated in 1833 was its recognition of the principle of interception. Longitudinal drains of large section were laid out parallel to the river and only 3 of the 40 old mouths of independent sewers were left in service, the remaining systems being made to discharge into the intercepters. The rain water falling on the roofs was taken at first through leaders to the gutters, but later was diverted in some cases to the large "house drains," with sections big enough for a man to walk through, connecting the houses with the sewers but used only for delivering waste water and not for excrementitious matter. The latter was discharged for many years into cesspools, one frequently serving for an entire block of houses.

[1] "One reason for making the smallest class of public sewers in Paris so much larger than they are in every other city is the practice which, till within 10 years, existed only there, of placing the water mains in them." E. S. Chesbrough, 1856. In commenting on the location of water mains in the sewers, Humblot stated in a report, in 1886, that the flat portions of Paris were largely on filled ground and the hills were undermined by old quarries, so that leaks in mains laid in earth would rarely be detected, and it was particularly desirable to keep the mains exposed so that their condition could be observed constantly. Telegraph and telephone lines and pneumatic tubes for transporting mail were placed in the sewers on account of the facility of installation and maintenance. Gas mains were also placed in a few of the sewers until explosions led to the abandonment of this practice.

9. Dry vs. Water-carriage Systems.—The Parisians committed the mistake, about 1820, of insisting by ordinance on cesspool construction. It was recorded that the whole subsoil of Paris was on the point of becoming putrid with cesspit matter, and that the ordinance was passed in consequence. The cesspools finally became so offensive that the nostrils of the Parisians were plagued and a new system of sewerage was developed. At that time European sanitarians were divided into two schools, advocating respectively the "dry" and the "water-carriage" methods of collecting excrementitous matter. In the former this matter is collected and removed in pails, and in the latter it is flushed into the sewers. The former is still used in a number of European cities, but is employed only on a small scale in the United States.

There are sections in many cities where it is impracticable to discharge sewage into the city sewerage system due to topographical obstacles difficult or expensive to overcome, to remoteness, or to lack of funds. Here the "dry" method is sometimes employed, but the use of cesspools is to be preferred if soil conditions are adapted to their use. The former is insanitary, inconvenient, expensive as to final disposal of excreta, and frequently a menace to public health through the spreading of disease germs by flies.

Tight cesspools require frequent removal of their contents and transportation to a place of disposal; and if not emptied in time, cause much trouble by overflowing. Leaching cesspools, which are intended to allow their contents to percolate into the surrounding earth, are often satisfactory if located in porous earth and at a sufficient distance from dwellings: but their use results in pollution of the soil and endangers well water supplies, and after a time the earth is likely to become clogged so that the contents of the cesspool will no longer percolate away.

The water-carriage system is sanitary and removes the household wastes immediately and economically to the point of disposal. It is the most satisfactory method in use at the present time and where it is at all practicable, should be used in preference to the "dry" system or cesspools.

10. Early American Sewers.—Little is known of the early sewerage works in the United States. Often they were constructed by individuals or the inhabitants of small districts, at their own expense and with little or no public supervision.

There was a tendency in this country, as elsewhere, to construct the early sewers of needlessly large dimensions. One of the oldest sewers in Brooklyn was in Fulton Street. Although it drained an area of less than 20 acres and was on a grade of 1 in 36, it was 4 ft. high and 5 ft. wide. For many years the largest sewer in Manhattan was that in Canal Street, built somewhere between 1805 and 1810; it was 8 by 16 ft. in section and about 1850 was in very bad condition, being referred to by engineers of that time as affording instructive information of things it was wise to avoid. Its large size doubtless was made necessary by the existence of a brook at this place which was at one time provided with plank walls and was used by small boats, as illustrated in Valentine's "Manual of New York." In some cases, the sewers were not only very large at their outlets but were continued of the same size to their heads; it was impossible to secure adequate velocity in such sewers unless they were laid on steep grades, and consequently some of them became offensive when the sludge accumulating in them underwent decomposition. In some cases, in fact, the slopes were in the wrong direction.

It should be noted that sewers were constructed originally, both here and abroad, for the removal of storm water, all excreta being excluded from the sewers of London till 1815; from those of Boston till 1833; and from those of Paris till 1880. In 1847, the connection of houses and cesspools to the sewers was required by law in London, while in Baltimore even as late as in 1922 there remained 20,000 houses unconnected with sewers.

The first application of engineering skill to the design of American sewers was in 1857, when Julius W. Adams was appointed to prepare plans for the sewerage of Brooklyn, N. Y. For many years thereafter the Brooklyn sewers served as models. In the following year E. S. Chesbrough submitted his first report on a comprehensive sewerage system for Chicago, Ill. In 1874, J. Herbert Shedd established the basic principles for the design of a sewerage system at Providence, R. I. Two years later a committee, consisting of Messrs. E. S. Chesbrough, Moses Lane, and Dr. C. F. Folsom, reported on the sewerage of Boston, Mass., advocating the general plan now in effect.

11. Introduction of Separate Sewers.—The United States suffered, just as England did at an earlier date, from the improper design of separate systems of sewerage in which the

house sewage and rain water are kept nearly or quite distinct. Just who designed the first system of sewers for removing house sewage separately is not definitely known, but the principle was strongly advocated as early as 1842 by Edwin Chadwick. He has been called the "father of sanitation in England," and unquestionably played an important role in arousing that country to the need of greater cleanliness, not only in cities but also in rural districts. He was a man of convincing address, great self-reliance, and enthusiasm, and strong imagination which was, unfortunately, not restrained by technical knowledge. As a result he advocated, even in meetings of engineers, so-called hydraulic principles and some features of design that were wholly incorrect, which at last resulted in his being publicly branded as a charlatan at a meeting of the Institution of Civil Engineers at which he was in attendance.

However, the principle of the separation of house sewage from rain water, advocated by Chadwick, was so meritorious for many places that it was developed along rational lines by a number of leading English engineers, notably Sir Robert Rawlinson, whose "Suggestions as to Plans for Main Sewerage, Drainage, and Water Supply," published by the Local Government Board, did much to prevent the laying of sewers of too small size and poor alignment, without proper facilities for the cleaning which is likely to be necessary in all such works.

The separate system received much study by American engineers, as was natural in view of their reliance on English practice for precedent. Fortunately, however, the difference between the character of the rainfall in England and the United States was known here and its influence on the design of sewerage works was appreciated. The English rains are more frequent but less intense, and hence our storm-water drains must be larger for like topographical conditions. Our heavier rains afford more vigorous flushing action in the sewers, so that the necessity for the rather elaborate provisions for flushing combined sewers in many European cities is not so evident here. Wherever the surface drainage could be cared for satisfactorily at a low cost without the use of large combined sewers receiving both sewage and rain water, there was a manifest advantage in adopting the separate system, providing only the sewers and leaving the building of storm-water drains for the future. This was done

at about the same time (1880) in designs prepared by Benezette Williams for Pullman, Ill., and George E. Waring, Jr. for Memphis. The Memphis system was the most conspicuous, although a comparative failure, a fact which the people of that city naturally suppressed for business reasons for many years.

By 1882 the main lines in some places were reported by the City Engineer, Niles Meriwether, to be taxed to their full capacity. The inadequate capacity of the larger sewers resulted in the construction of a relief sewer during 1885–1886. Engineers familiar with the conditions were convinced that some of Colonel Waring's favorite details had proved defective, and that the Rawlinson type of separate system, with larger pipes laid without vertical or horizontal bend between successive manholes, was preferable. The partial failure of the so-called Waring system was demonstrated, therefore, in about 5 years' experience at Memphis; this was a little longer than was required to demonstrate the same thing at Croydon, England, 30 years before the Memphis experiment. The National Board of Health felt some distrust regarding such systems soon after its formation, and accordingly it sent Rudolph Hering to Europe on a tour of investigation, which lasted nearly a year. On his return he prepared an elaborate report on the principles of sewerage and their exemplification in the best works of Europe, which outlined the respective fields of the separate and combined systems.

12. Development of Methods of Sewage Treatment.—The disposal of the sewage of most cities, until recent years, was carried out by the easiest method possible, without much regard to unpleasant conditions produced at the place of disposal. Irrigation with sewage was apparently practiced at ancient Athens, but there is very little definite information on any methods of disposal on land down to about three hundred years ago, when sewage farming was successfully introduced at Bunzlau, Germany. The earliest municipal work of the kind in Great Britain was on the Craigentinny meadows of about four hundred acres extent, receiving the sewage of a part of Edinburgh for about a century. The subject of disposal received only occasional local attention, however, until the construction of sewerage systems after the cholera epidemics of 1832–1833 and 1848–1849. Owing to the small size of British streams, their pollution by the sewage discharged into them soon became

a nuisance. Interference with agricultural and manufacturing uses of water was apparently at first given more attention than possible danger to health. When the cholera epidemic of 1854 had been suppressed, Parliament passed the comprehensive Nuisances Removal Act of 1855, to which reference has already been made. This did not make sewage treatment compulsory, however, nor did the Rivers Pollution Prevention Act of 1876, although as early as 1865 a royal commission had reported:

First, that whenever rivers are polluted by a discharge of town sewage into them, the towns may reasonably be required to desist from causing that public nuisance.

Second, that where town populations are injured or endangered in health by a retention of cesspool matter among them, these towns may reasonably be required to provide a system of sewers for its removal.

In 1880 the discovery of the bacillus of typhoid fever, by Eberth in Germany, marked the beginning of a new era in sanitation. Previously, the relation of pollution to disease had been but faintly understood, as the science of bacteriology was in its infancy, and its application to matters of stream pollution and sewage disposal had not been grasped. In 1877 Shloesing and Muntz, in France, and, in 1882, Robert Warington, in England, proved conclusively that the oxidation of ammonia and organic matter was effected by the agency of living organisms, and Warington proceeded to devise practical methods whereby living organisms could be utilized for the nitrification of the organic matters in sewage. Later, through the studies at the Lawrence Experiment Station of the Massachusetts State Board of Health, the fundamental biological conditions underlying the oxidation processes of sewage treatment became established.

Two methods of treating sewage had been in vogue before this time. The irrigation of land by sewage was the older of these, but the precipitation of the solids and some of the dissolved matter by chemical treatment and subsequent sedimentation attracted more attention owing to its exploitation by promoters as well as to the favorable opinion of it held by many careful and conservative engineers.

The density of population in England and the very small amount of land well suited for sewage farming and filtration led to particular interest in intensive methods of treatment, whereby in plants of comparatively small area the sewage was rendered suitable for a final treatment on land, which was

practically compulsory for most English systems discharging into fresh water. This constraint was exercised by the Local Government Board, without whose approval money could not be raised for public works except by special act of Parliament; the Board required a final land treatment until comparatively recently. Consequently septic tanks, trickling filters, and contact beds, which were rapidly developed after the underlying biological factors had been determined, were received with acclamation and tested on a practical scale that was unwarranted, for instance, in Germany.

The disposal of sewage in the United States did not receive so much attention 40 years ago as in England, because the extent of the nuisance caused by its discharge into the relatively large bodies of water was not so marked and because of the greater area of land suitable for broad irrigation or intermittent filtration on beds graded in situ, and of relatively cheap materials suitable for the construction of artificial treatment beds in some localities where pollution was objectionable. Its importance was foreseen by the Massachusetts State Board of Health early in the seventies, and its secretary, Dr. C. F. Folsom, made a careful study of disposal in Europe, which resulted, in 1876, in a report which was the most complete statement that had been made of the state of the art at that time. Irrigation and filtration were introduced in a few places, but it was not until certain rivers in Massachusetts became quite offensive that any work on a large scale was undertaken. The first extensive treatment plant utilized chemical precipitation and was built at Worcester, Mass., in 1889–1890, from the plans of Charles A. Allen with the advice of James Mansergh of London and Prof. Leonard P. Kinnicutt of Worcester. It was about this time (1887), that the Massachusetts State Board of Health, which had been given large powers of control over the disposal of sewage, established the Lawrence Experiment Station for the study of both water and sewage treatment; the influence of the research work done there has been deep and far-reaching, as above noted, being particularly noteworthy for the prominence given in early years to intermittent filtration, a method of disposal neglected in England on account of the limited tracts of land suitable for its practice. The increasing demand for sewage treatment and the impracticability of procuring sufficient areas of suitable soil for land treatment in many localities,

particularly for cities even of moderate size, have led to the rather wide adoption in the last quarter-century of more intensive methods of treatment.

13. Respective Usefulness of Separate and Combined Sewers. A combined sewerage system requires but a single conduit in a street, thus saving a portion of the space required by the two conduits of the separate system. This is important in streets congested with underground structures. Usually the quantity of domestic and industrial sewage is not greater than the margin of error in estimates of the quantity of storm water to be removed. Therefore the inclusion of the sewage with the storm water does not require a combined sewer appreciably larger than the storm-water drain required for serving the same district adequately. Even where an appreciably larger section is required, the additional cost of building the combined sewer as compared with the cost of the drain will generally be practically negligible. Hence the cost of a combined system is less than that of both a separate sewer system and a storm drainage system, where complete underground removal of storm water and sewage is necessary.

It may be undesirable to allow storm water from urban districts to contaminate neighboring bodies of water, not only because of sanitary objections but sometimes because of the disagreeable appearance of accumulations of refuse or deposits of sediment forming on the bottoms of these water courses, or of floating matter and slick. Such conditions in many cases make necessary an extensive drainage system which might as well be used to remove sewage, thus avoiding the additional expense of dual systems.

Where a separate system of sewers is paralleled by a drainage system and roof water cannot be allowed to flow over the surface of the ground, a system of collecting drains is required outside the buildings in addition to the usual inside plumbing, or two systems of plumbing are necessary within the buildings, one to take care of the sewage and the other to remove the roof water. This is not only expensive to the property owner but such systems offer opportunities for mistakes in making connections which need only be few in number to defeat the entire purpose of the separate systems. There have been sewerage and drainage systems administered so badly that the surface water was discharged through incorrectly made connections into separate sewers, forcing the sewage back into the cellars

and out through manholes upon the streets; in other cases sewage has been discharged into drains, resulting in unnecessary and objectionable pollution of waters into which they emptied.

Notwithstanding these disadvantages of the separate system there are certain conditions under which it may be advantageous. Dr. Hering summarized[1] some of them as follows:

The separate system is suitable,

Where rain-water does not require extensive underground removal and can be concentrated in a few channels slightly below the surface, or where it can safely be made to flow off entirely on the surface. Such conditions are found in rural districts where the population is scattered, on small or at least short drainage areas, and on steep slopes or side hills.

Where an existing system of old sewers, which cannot be made available for the proper conveyance of sewage, can yet be used for storm-water removal.

Where purification is expensive, and where the river or creek is so small that even diluted sewage from storm-water overflows would be objectionable, especially when the water is to be used for domestic purposes at no great distance below the town.

When pumping of the sewage is found too expensive to admit of the increased quantity from intercepting sewers during rains, which can occur in very low and flat districts.

Where it is necessary to build a system of sewers for house drainage with the least cost and delay, and the underground rain-water removal, if at all necessary, can be postponed.

The principle of separation, although often ostensibly preferred on sanitary grounds, does not necessarily give the system in this respect any decided advantage over the combined, except under certain definite conditions. Under all others, preference will depend on the cost of both construction and maintenance, which only a careful estimate, based on the local requirements, can determine.

GENERAL FACTORS AFFECTING THE SEWERAGE PLAN

The general plan of a sewerage system is governed by two prime factors, the topography of the city and the place of disposal of the sewage. The two are sometimes so simple in their effect that the general arrangement to be followed is self-evident, but in other cases they have complex inter-relations which require protracted study before the best system can definitely be determined.

14. Influence of Topography on Sewerage Plan.—The arrangement of the small and large sewers which make up a sewer system is influenced largely by the topography of the city.

[1] Report to National Board of Health on European Methods of Sewerage and Sewage Disposal, 1881.

In a large city situated on a flat plain without any neighboring rivers or lakes into which the sewage can be discharged without elaborate treatment, the radial system may prove best. This has its most elaborate development in Berlin, where it was introduced by Hobrecht. The city is divided into a number of sectors and the sewage of each sector is carried outward by pumping to its independent disposal farm, or the trunk sewers of two or more sectors may be connected to a farm. The advantage of this system is that most of the sewers are likely to be of adequate capacity for a long period, and the large expensive sewers are reduced to their minimum length.

In most cases such an arrangement is rendered impracticable by the configuration of the ground surface. Usually, moreover, old sewers complicate the problem, for it is always desirable to utilize existing structures so far as practicable. Only in rare cases does the engineer have an opportunity to design a complete sewerage system for a large city, as was the case in New Orleans and in Baltimore.

In Baltimore, where the sewage had to be taken 5¾ miles outside the city for treatment, and where there was no objection to the discharge of storm water into the nearest water courses adapted to receiving it, separate systems of sewers and drains were built. The city is intersected by four streams, which discharge into branches of the Patapsco River. One of these streams received so much foul run-off that it has been covered over; the others are open. The Patapsco and its branches are tidal arms of Chesapeake Bay. The drainage area was divided into 28 districts, and the storm-water drains in each one were planned independently of the rest, to fit the topography and arrangement of streets in the best way. These drains were kept as close to the surface as possible, in the interest of economy and in order not to force the sewers so low that it would be difficult to connect the houses with them. In one low-lying district where the drainage problem was particularly difficult, the plans called for raising the street grade and building a drain to carry the storm water into the Patapsco River instead of a nearer stream which was likely to have its surface raised considerably during floods, a condition which might cause a surcharge of the drains emptying into it.

The removal of the house sewage was a much more complicated problem. Part of it comes from districts which are high enough

to enable the sewage to flow by gravity to the treatment works, but a large part has to be pumped. The dividing contour line between these two sewerage districts was determined by two factors, the elevation at which the sewage must be discharged at the treatment works and the minimum safe slope of the outfall sewer from the city to the works. The accompanying plan of the main sewers, Fig. 1, shows where the outfall sewer reaches the eastern boundary of the city and is continued through

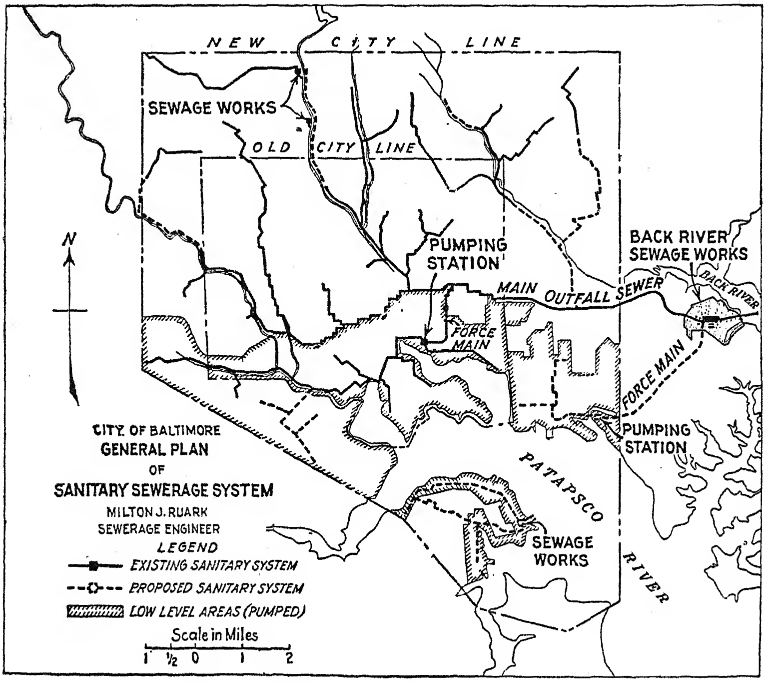


FIG. 1.—Baltimore main sewer system.

it toward the western boundary as a high-level main, receiving all the sewage that can be delivered by gravity to the disposal works. The sewage of the low-lying portions of the city is collected by four main sewers, on two of which are small pumping plants to lift the sewage to avoid excessive depth of trench, which is undesirable on account of the high cost of construction in deep trenches in water-bearing soil, and the difficulty of connecting the tributary sewers satisfactorily with deep-lying mains. All these mains run to a station containing five pumps,

each with a nominal rating of 27,500,000 gal. a day against a total head of 72 ft. These pumps force the sewage through two lines of 42-in. cast-iron mains 4,550 ft. long into a sewer about 1 mile long, discharging by gravity into the main outfall sewer.

Between the arrangement of sewers in the Borough of Manhattan, formerly discharging both storm water and house sewage through short lines into the nearby rivers through many outlets, and the arrangement at Baltimore with its separation of the storm water and the sewage, its high and low levels, pumping stations, long outfall sewer and elaborate sewage treatment works, there is an infinite variety of combinations practicable. In every case, however, the topography suggests the natural drainage and the street plan exercises a more or less strong modifying influence. Special attention should be paid to the low-lying districts, for it is there that the largest sewers must be built in many cases, and the difficulties of construction are the greatest. It may be found advisable to reduce such work to a minimum by constructing an intercepting sewer at a somewhat higher level and thus restrict the construction in the low-lying sections to small sewers only deep enough to serve the property of those districts.

Another influence of topography on sewerage plans, often overlooked, was stated as follows by Dr. Hering in his report of 1881 to the National Board of Health:

> In case of sudden showers on a greatly inclined surface which changes to a level below, the sewers on the latter will become unduly charged, because a greater percentage flows off from a steeper slope in a certain time. To avoid this uneven reception, the alignment should, as much as possible, be so arranged as to prevent heavy grades on the sloping surface, at the expense of light ones on the levels. In other words, the velocity should be equalized as much as possible in the two districts. This will retain the water on the slopes and increase its discharge from the flat grounds, thus corresponding more to the conditions implied by the ordinary way of calculating the capacity of sewers. It will therefore become necessary not to select the shortest line to the low ground, but, like a railroad descending a hill, a longer distance to be governed by the gradient. This does not necessarily imply a longer length of sewers for the town, because more than one sewer for a street is not required by it.

Still another decided influence of topography is shown where the configuration and surroundings of the city are such that it is advisable to employ combined sewers in all parts of the city down to the lowest contour line which will permit storm-

water overflows to be used. This is the rule adopted by E. J. Fort for the new sewerage works of Brooklyn. Below this contour line, the combined sewers become storm-water drains, run at a higher level than the separate sewers, so as to have a free outlet to tide water, and the sewage of the low districts is pumped to points of disposal.

After the most favorable location of the sewers has been determined from an exclusive sewerage viewpoint, the desirability of minor changes of position to avoid needless interference with travel through busy streets should receive attention. The construction of a sewer in a narrow or crowded street costs the community a considerable sum in indirect damages and directly affects those having places of business on the streets.

15. The Necessity for Sewage Treatment.—The main difficulty in sewage disposal is due to the putrescible organic substances present in the sewage and in that portion of the street wash which flows away during the early part of a rain. These substances necessitate treatment of the sewage to prevent objectionable conditions in many instances. While these deleterious organic substances are small in amount—generally considerably less than one-tenth of 1 per cent, the remaining 99.9 per cent of the sewage being water—their successful handling and treatment constitutes the essential problem in sewage disposal.

The part of the street wash which enters the sewers and drains after the refuse on the pavements has been carried off is relatively much less polluted, and on this account it can often be discharged into waters or over land unfitted to receive seriously polluted water. Where the streams into which sewage must be discharged are small and it is necessary to keep them clean, as is generally the case in Great Britain, the chief objection to the discharge of storm water into them is due to the suspended solids carried by the flowing water. These may form undesirable deposits on the banks and the bed of a stream and the refuse floating on the surface may give an objectionable appearance to the water. Consequently it is the general engineering rule in Great Britain that where the dry-weather sewage must be treated before it is discharged into a body of water, the storm flow must also be treated before discharge into the same body of water, until its volume equals at least three times the normal flow of dry-weather sewage. Many American cities with com-

bined sewers now permit the discharge of untreated storm flows into neighboring rivers or lakes, and it is not unlikely that some of them will be faced with the necessity of installing works for removing part of the suspended matter from the storm flows before final discharge. Even where separate storm drains are provided such treatment may be necessary in some cases.

The necessity for a large part of the treatment which sewage should receive before its final disposal is due to the organic matter it contains. Under normal conditions, this changes fairly rapidly into other substances, some of them offensive, and the change is accompanied by a rapid growth of bacteria. Some species of bacteria originally present may be objectionable in the water receiving the sewage. The purpose in treating sewage is to remove the solids which cause deposits in the rivers and ponds, as well as undesirable surface conditions, and to carry out certain changes in the organic matter without causing offense, under the control of the operator of the treatment works, until the final effluent from the works has its organic matter in such a state that any further changes which it may undergo can proceed in the river or lake without detriment to the quality of the water for the purposes for which it is used.

There are many different methods of treatment, such as dilution in a large body of water, removing the solids by screens or by sedimentation in small or large tanks, allowing the sewage or the organic solid matter settling from it to remain in tanks under conditions which will cause various changes in it, changing the character of sewage by aeration, and filtering the sewage or the effluent from some previous form of treatment. In each of these methods of treatment a certain minimum fall through the tanks or filters and the outfall sewer is necessary and must be provided, naturally or by pumping, and consequently the method of treatment and disposal of sewage affects not only the direction from the city which the outfall sewer or sewers must take but also the slopes of such sewers.

16. Influence of Method of Disposal on Sewerage Plan.—There are three general methods of disposal which affect the design of a sewerage system.

One of these disposal methods is to discharge directly into a river or other body of water on the shore of which the city lies. The Borough of Manhattan, New York City, arrangement prior

to about 1922 shown in Fig. 2, was an example of this, with its numerous main sewers running east and west to outlets at the Hudson and East Rivers.

Another method of disposal is to intercept the sewage and convey it to a point in the adjoining body of water where it

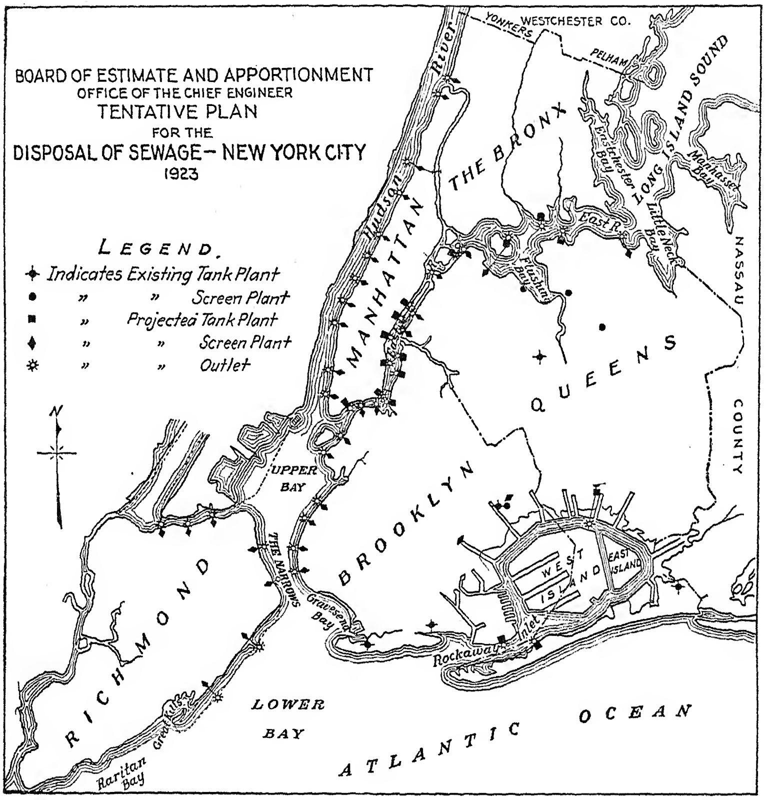


FIG. 2.—Locations of sewer outlets in 1922, New York City.

will not cause trouble; this may not be necessary at first, but in most cases it is inevitable if the city grows as rapidly as do most American municipalities, and attention must be paid to it, particularly to the future desirability of separating the sewage from part of the storm water. The Cleveland system shown in Fig. 3 is an example of this intercepting plan, although some of the sewage is now being treated before its discharge.

A third method of disposal is to treat the sewage so as to materially change its character before it is discharged into a body of water. This makes it necessary to deliver the sewage to treatment works, suitable sites for which are difficult to procure in many cases, particularly where the country is well built up and enough open land properly situated is not available in the city, while neighboring towns object to the plant being

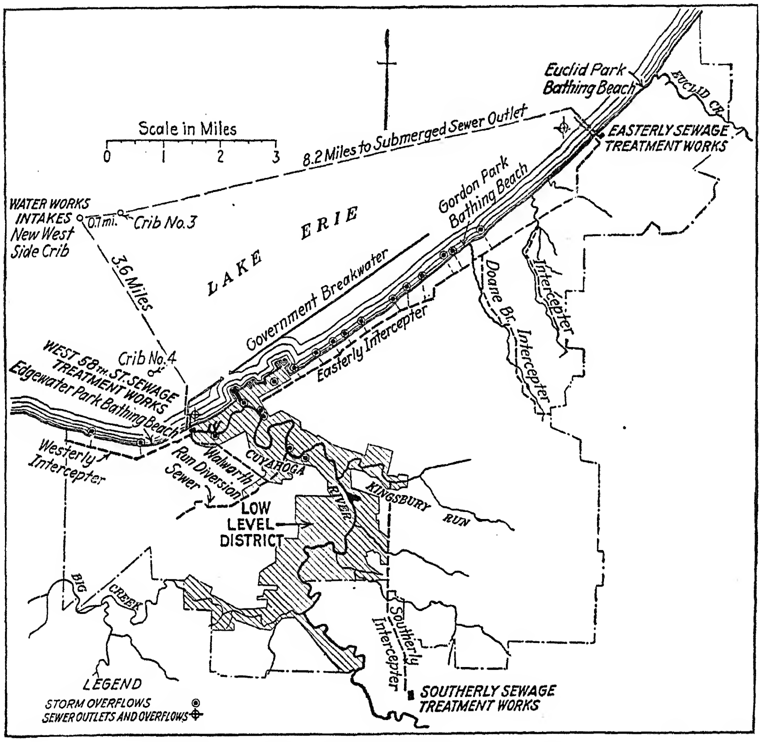


FIG. 3.—Cleveland intercepting sewer system.

located within their limits. The separation of storm water from the sewage often becomes financially advisable, so as to permit the former to be discharged by short, direct lines into the river, lake or bay nearby. In the case of combined sewers, the same end is attained by making provision at one or more points for the discharge of the storm water in excess of a predetermined amount, through overflow weirs or chambers into channels or other outlets leading directly to the river or lake.

In cities situated on rivers it is customary to convey the sewage to a point below the thickly settled district, whether the sewage is to be discharged in its raw state or is to be treated by some artificial means before discharge. This plan avoids the danger of causing objectionable conditions along the water front and, perhaps, the contamination of the local water supply by the discharge of sewage into the river. Similarly, cities situated on the Great Lakes may convey their sewage in the direction of the trend of the waters to a point well beyond the populous districts. This policy is not so well established nor so effective as in the river cities, for the lacustrine current is not so marked as most river currents and it may be altered temporarily by adverse winds. Cities situated on tidal waters often build long intercepting sewers to convey the sewage to favorable localities for discharge.

Each method of treatment has certain requirements which may determine the location at which it can be practiced. Intermittent sand filtration requires large areas of suitable sand. Rapid filtration through deep beds of coarse aggregates requires more head than other methods. Aeration requires a power plant to supply compressed air. Some tank treatments are accompanied by offensive odors. These conditions, explained in later chapters, make it desirable to study the practicability of discharging sewage at several nearby points, after suitable treatment to fit it for the conditions at each outlet, rather than the delivery of all of the sewage through a long, expensive outfall sewer to a single treatment plant. It should be understood at the beginning of any study of sewage disposal that the purpose of treatment is to protect the water or land receiving the sewage. Changing the character of the sewage merely for the sake of making it less offensive or dangerous is a waste of money unless it is necessary, just as it is a waste of money to provide works to carry the sewage farther than the local conditions in each case require. It should also be understood that where the conditions warrant treatment by dilution, which means the discharge of sewage into a large body of water, that method of treatment is not only the cheapest in first cost in most cases but is just as well established as a truly scientific process as the most elaborate artificial treatment by sedimentation, aeration, filtration and sterilization, needed by a city not favored by Nature with conditions admitting the use of less expensive methods.

17. Municipal Liability for Inadequate Sewers.—There is a quite general belief that a city is never liable for damages caused by the inadequate capacity of sewers. As a matter of fact, a city may be liable for such damages under certain conditions, and even where there is no legal liability a grave moral responsibility rests upon the engineer who designs a sewerage system. The present legal aspect of the subject places the engineer upon a strictly professional plane like that of a physician or lawyer, and it is one of the few instances in which the courts have recognized the professional nature of engineering.

It has been held that if a city has a collection of sewers and it cannot prove that this collection was built or rebuilt in accordance with a comprehensive plan prepared by an engineer presumably competent to design such works, the city is liable for any damages which may result from the inadequate capacity of the sewers.[1] This makes it advisable for the city to employ competent engineers to design its sewers and to approve the sewers in any additions incorporated within the municipal boundaries from time to time.

In some cases the courts have held that if a city employs a presumably competent engineer to design its sewerage system and the construction proceeds in conformity with his plan and subsequent modifications of it by presumably competent engineers, it is not liable for damages due to a fault in the plan. In Massachusetts, it is not clear that the sewer system must be designed by an engineer, the law being stated[2] as follows: "Certain governing principles of law are well established. A municipality is not responsible for any defect or want of efficiency in a general plan for drainage adopted by a board of public officers, even though private individuals may be exposed to great inconvenience and loss." The city is liable, however, for damages due to the known lack of repair of sewers or its failure to maintain them to the standard of efficiency of the original construction.

The damages caused by inadequate sewers are generally due to an insufficient estimate of the quantity of sewage for which provision must be made, particularly where the combined

[1] A typical ruling of this sort was made in the case of Hart *vs.* City of Neillsville, 104 N. W. Rep. 699, where the Wisconsin Supreme Court made the decision.

[2] Diamond *vs.* North Attleborough, 219 Mass. 590; see, also, 4 N. E. Rep. 321 (New York Court of Appeals); 61 Atl. Rep. 180 (Maine Supreme Court).

system is used. Too large capacity not only causes needless expense but develops a tendency for the sewage to flow so slowly and in such shallow streams that part of the suspended solids will settle on the inverts, forming offensive deposits. If the sewers are too small, cellars may be flooded with sewage, which may even rise from manholes and gutter inlets and flow over the surface of the ground, resulting in damage and litigation. The subject, in some respects, is one of the most important in the field of sewerage engineering, one calling for sound judgment as well as knowledge.

SEWERAGE OF BUILDINGS

18. The Plumbing System.—Since the house plumbing system is the beginning or upper end of the sewerage system, it is desirable to consider it briefly before entering upon the detailed study of the sewers themselves.

The *plumbing system* of a building includes the *pipes* for the water supply, the *fixtures* for its use and the *drains* for its removal. The *drains* sometimes carry rain water from the roof when the city permits its discharge into the sewer. The names of the various pipes and fixtures are shown on Fig. 4.

19. Relation of Plumbing to Health.—As the house drainage system carries putrescible material sometimes containing pathogenic or disease producing bacteria, it is necessary that it be both water and air tight so as to prevent the leakage of sewage or foul air and gases into the building. Such leakage yields unpleasant odors and makes possible the infection of food through insect carriers. Leakage in the soil outside the building may contaminate the water in adjacent wells. Gases given off from decomposing feces, fats and other household wastes or from gasoline from garages, are common in sewers and may enter a building through leaky or unsuitably constructed plumbing. Illuminating gas may leak into sewers, and the processes of decomposition produce hydrogen sulphide, carbon dioxide, methane and other gases. While there is slight probability of the presence of bacteria in sewer gases, they are malodorous and may produce headache, nausea and fatigue. It is evident, therefore, that the plumbing system should be suitably constructed to provide for convenience and the protection of health, although the latter has often been overemphasized.

20. Basic Plumbing Principles.—The U. S. Bureau of Standards[1] has formulated the basic principles to be observed in all plumbing installations as follows:

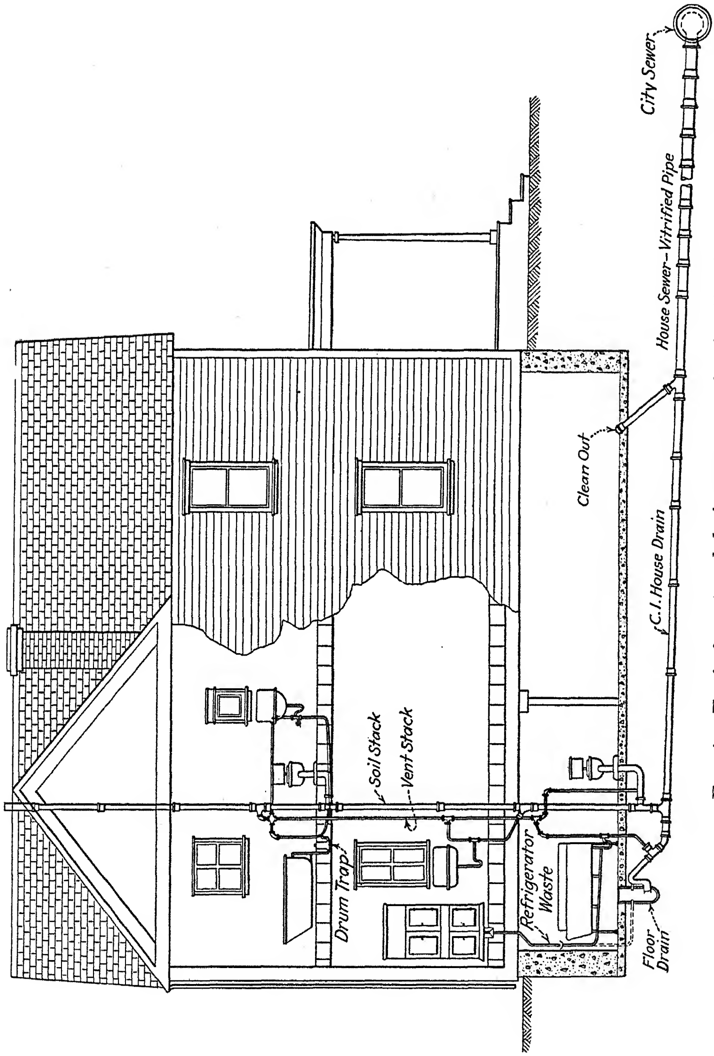


Fig. 4.—Typical vent and drainage system for dwelling.[2]

1. All premises intended for human habitation or occupancy shall be provided with a supply of pure and wholesome water.

[1] "Recommended Minimum Requirements for Plumbing in Dwellings and Similar Buildings." 1924, p. 13.

[2] Adapted from Wisconsin State Plumbing Code.

2. Buildings in which water-closets and other plumbing fixtures exist shall be provided with a supply of water adequate in volume and pressure for flushing purposes.

3. The pipes conveying water to water-closets shall be of sufficient size to supply the water at a rate required for adequate flushing without unduly reducing the pressure at other fixtures.

4. Devices for heating water and storing it in "boilers," or hot-water tanks, shall be so designed and installed as to prevent all dangers from explosion and also prevent a back flow of hot water through a meter connected with a public water supply.

5. Every building intended for human habitation or occupancy on premises abutting on a street in which there is a public sewer shall have a connection with the sewer, and, if possible, a separate connection.

6. In multiple dwellings provided with a house drainage system there shall be for each family at least one private watercloset.

7. Plumbing fixtures shall be made of smooth non-absorbent material, shall be free from concealed fouling surfaces, and shall be set free of inclosures.

8. The entire house drainage system shall be so designed, constructed, and maintained as to conduct the waste water or sewage quickly from the fixture to the place of disposal with velocities which will guard against fouling and the deposit of solids and will prevent clogging.

9. The drainage pipes shall be so designed and constructed as to be proof for a reasonable life of the building, against leakage of water or drain air due to defective materials, imperfect connections, corrosion, settlements or vibrations of the ground or building, temperature changes, freezing, or other causes.

10. The drainage system shall be provided with an adequate number of cleanouts so arranged that in case of stoppage the pipes may be readily accessible.

11. Each fixture or combination fixture shall be provided with a separate, accessible, self-scouring, reliable water-seal trap placed as near to the fixture as possible.

12. The house-drainage system shall be so designed that there will be an adequate circulation of air in all pipes and no danger of siphonage, aspiration, or forcing of trap seals under conditions of ordinary use.

13. The soil stack shall extend full size upward through the roof and have a free opening, the roof terminal being so located that there will be no danger of air passing from it to any window and no danger of clogging of the pipe by frost or by articles being thrown into it or of roof water draining into it.

14. The plumbing system shall be subjected to a water or air pressure test and to a final air-pressure test in such a manner as to disclose all leaks and imperfections in the work.

15. No substances which will clog the pipes, produce explosive mixtures, or destroy the pipes or their joints shall be allowed to enter the house-drainage system.

16. Refrigerators, ice boxes, or receptacles for storing food shall not be connected directly with the drainage system.

17. No water-closet shall be located in a room or compartment which is not properly lighted and ventilated to the outer air.

18. If water-closets or other plumbing fixtures exist in buildings where there is no sewer within reasonable distance, suitable provision shall be made for disposing of the house sewage by some method of sewage treatment and disposal satisfactory to the health authority having jurisdiction.

19. Where a house-drainage system may be subjected to back flow of sewage, suitable provision shall be made to prevent its overflow in the building.

20. Plumbing systems shall be maintained in a sanitary condition.

21. The Water Supply.—To determine the pipe sizes required for the water supply system, it is necessary to know the amount to be supplied to the various fixtures. The Bureau of Standards[1] has suggested that fixtures be rated upon a *fixture unit* basis, this unit being 1 cu. ft. per min. or the rate of discharge from an ordinary wash basin with a 1¼ in. outlet. The following values for the various fixtures are recommended by the Bureau.

	Fixture Unit
Lavatory or wash basin	1
Kitchen sink	1½
Bath tub	2
Laundry tray, urinal, shower bath or floor drain	3
Slop sink	4
Water closet	6

[1] "Recommended Minimum Requirements for Plumbing in Dwellings and Similar Buildings," 1924.

With the discharge rate known, the diameter of pipe is computed by any of the accepted hydraulic formulas.

Table 1 gives the approximate quantities of water to be supplied to the various fixtures with the necessary pipe sizes, as suggested by W. S. Timmis.[1]

TABLE 1.—RECOMMENDED SIZES OF WATER SUPPLY PIPES TO FIXTURES
(Standard wrought pipe)

Fixture	Number of fixtures								
	1	2	4	8	12	16	24	32	40
Water closet									
Tank type—g.p.m.	8	16	24	48	60	80	96	128	150
Pipe size, in.	½	¾	1	1¼	1½	1½	2	2	2
Flush valve type—g.p.m.	30	50	80	120	140	160	200	250	300
Pipe size, in.	1	1¼	1½	2	2	2	2½	2½	2½
Urinal									
Tank type—g.p.m.	6	12	20	32	42	56	72	90	120
Pipe size, in.	½	¾	1	1¼	1¼	1¼	1½	2	2
Flush valve type—g.p.m.	25	37	45	75	85	100	125	150	175
Pipe size, in.	1	1¼	1¼	1½	1½	2	2	2	2
Wash basin[1]									
G.p.m.	4	8	12	24	30	40	48	64	75
Pipe size, in.	½	½	¾	1	1	1¼	1¼	1½	1½
Bathtub									
G.p.m.	15	30	40	80	96	112	144	192	240
Pipe size, in.	¾	1	1¼	1½	2	2	2	2½	2½
Shower bath									
G.p.m.	8	16	32	64	96	128	192	256	320
Pipe size, in.	½	¾	1¼	1½	2	2	2½	2½	3
Sinks[1]									
Slop, kitchen									
G.p.m.	15	25	40	64	84	96	120	150	200
Pipe size, in.	¾	1	1¼	1½	1½	2	2	2	2½

[1] Each faucet.
Sizes based on pressure drop of 30 lb. per 100 ft.
Hot-water faucets to be disregarded when estimating sizes of risers and mains.

22. The Drainage System.—The vertical drains carrying the discharge from fixtures are called *soil* or *waste stacks*, the former term being employed where the pipe carries the discharge of water closets. The horizontal runs are called *drains* and are laid on slopes of from ⅛ to ½ in. per foot. A *trap* is a depressed portion of the drain constructed so as to remain filled with water to prevent the free passage of sewer air, insects or vermin through the fixture into the room. Plumbing regulations require that all fixtures be trapped. Traps are usually *vented* as indicated in Fig. 4, to prevent their being siphoned or emptied by their

[1] TIMMIS, W. S.: *Journal*, Am. Soc. Heating and Ventilating Engineers, 1922; **28**, 397.

own discharge or that of other fixtures. Vents have a minimum diameter of 1¼ in., the required size of the main vents or vent stacks being determined by the size of the soil or waste stack, the number of fixture units served and the length of vent or vent stack required.

The problem of computing the size of soil stack needed for any particular case is more difficult than the computation for the water supply pipe sizes because the material to be carried lacks homogeneity. This material is a mixture of air, water and solids and while the solids only slightly affect the pressures produced in the stack, the air and water mixture undergoes numerous changes in volume and arrangement in the stack section, accompanied by rapid fluctuations in pressure. The matter is too complex for consideration here and the student is referred to the report of the Sub-Committee mentioned above for a more detailed discussion. The passage of a slug of water down the stack produces positive pressures or pressures greater than atmospheric at fixtures connected to the stack below the slug, while negative, or less than atmospheric pressures, are produced after the slug has passed. When it is realized that numerous slugs often follow each other in rapid succession, it is apparent that the pressure exerted on a given vent at any moment is difficult to ascertain. The pressure thus exerted at unvented traps is of the magnitude indicated by the formula $P = kH^{5/2}$, where P is the pressure at the trap, k is a constant depending upon piping details and the units used for expressing pressure or discharge, H is the height of fall to the fixture at

TABLE 2.—RECOMMENDED SIZES OF SOIL AND WASTE STACKS

Number of fixture units	Number of water-closets or equivalent	Diameter of stack, in.	Maximum permitted length, ft.
1	....	1¼	45
2 to 8	....	1½	60
9 to 18	....	2	75
19 to 36	....	2½	105
37 to 72	1 to 12	3	150
73 to 300	13 to 50	4	225
301 to 720	51 to 120	5	300
721 to 1,080	121 to 180	6	400
1,081 to 1,920	181 to 320	8	600

which the pressure is to be determined. Or, if the relationship between pressure and discharge is desired, the formula may be written $P = kQ^{5/2}$, where Q is the discharge rate down the stack. In these formulas the exponent, given as 2.5 for a trap having no vent, is reduced when vents are provided and may be as low as 0.33 for a completely vented trap.

The sizes of stacks recommended by the Bureau of Standards are given in Table 2. Where roof water is admitted to the house sewer, 180 sq. ft. of roof surface is taken as equivalent to one fixture unit.

CHAPTER II

QUANTITY OF SEWAGE

In this chapter the term "sewage" will be restricted to the liquids which are ordinarily discharged by *separate sewers*, comprising (1) water derived primarily from the public water supply and containing domestic and industrial wastes, (2) water derived primarily from privately owned water supplies and containing industrial wastes, and (3) infiltration.

The quantity of sewage to be removed from any community depends upon the population and the per capita contribution of sewage. The population of an urban community having undeveloped areas or one which may increase in area by annexation, will ordinarily increase with time and it is wise to make provision for some growth when designing sewerage works.

23. Period of Design.—The fulfillment of a sewerage project of any considerable size requires much time and the expenditure of large public funds, and the construction often causes material public inconvenience, so that in general it will prove economical to so design the main structures that they will be capable of serving the community adequately when its development and population shall have equalled those estimated for a time some years or decades after the completion and the beginning of operation of the works. The number of years from the date of design to the estimated date when the conditions of design will be reached is the "period of design," sometimes called the "economic period of design."

Laterals and submain sewers are usually designed to serve their tributary areas when completely developed, or for conditions estimated to be reached far in the future, as the slight additional cost for increased sizes of small pipes is usually immaterial compared to the total cost of the works. Large sewers, pumping stations and treatment works are usually designed for a comparatively short period of time, since these structures can generally be relieved or supplemented by parallel sewers or additional units. Many conditions may influence

the growth of a community and estimates of future requirements are attended by much uncertainty. It is, therefore, usually inadvisable to predicate designs for main sewerage and drainage works upon estimates of conditions assumed to exist more than from 30 to 50 years in the future. If the exact conditions of development, rate of growth, requirements, rates of interest and construction and operating costs could be exactly determined for the future, it would be possible to compute an "economic period of design." As these items cannot be determined with exactness, the engineer must determine the period of design by the exercise of his judgment.

24. Forecasting Increase of Population.—The increase in the population of a city is sometimes affected by exceptional conditions, which may be temporary, as was the case in Washington during 1917 and 1918, or permanent, as where a new industry develops phenomenal growth, like the rubber industry in Akron or the automobile industry in Detroit. Sometimes, as at Gary, Ind., practically a new city develops in a few years about a great industry. But in the usual case, the past records of growth of a city and the records of the growth of similar places furnish fairly reliable guides for estimates of future development for periods of 25 to 40 years.

The following methods of forecasting population growth may be applied to urban communities in the United States. Their application in studying problems in foreign countries should be made with caution as some factors which apparently are almost negligible in this country, such as racial characteristics, the social organization and climate, may be found to exert powerful influences on the growth of foreign communities.

The various available records may be used in any one of four ways, according to the assumptions made regarding the rate at which the population increases. These four assumptions are:

1. *Assumed Uniform Percentage Rate of Growth.*—When cities are young and thriving, it often happens that their rate of growth is high for some years, but an assumption that this percentage rate will persist for a long period will certainly lead to an over-estimate of the population in most cases. The assumption of a uniform percentage rate of increase is apparently most reliable in the case of old, large cities not subject to periods of great commercial or industrial activity.

2. *Assumed Curvilinear Rate of Growth.*—The information furnished by diagrams of the past growth of cities is very instructive, but an attempt to predict the future growth of a city from its past development alone, by extending the curve of that development, is likely to give misleading results, as will be shown later. Diagrams have a useful place in the study of changes in population, but they are not a substitute for an investigation of the various influences which have affected the city's growth in the past and may affect it in the future.

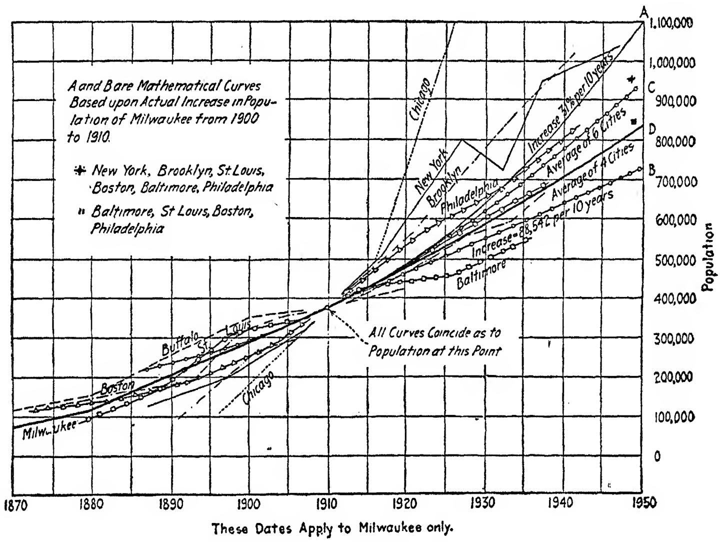
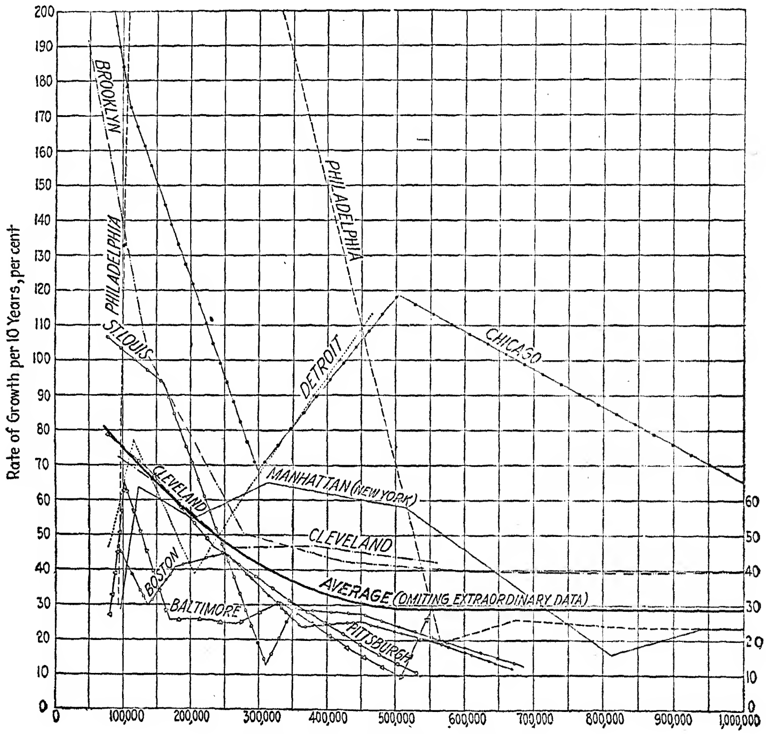


FIG. 5.—Growth of large American cities.

3. *Assumed Arithmetical Rate of Increase.*—Occasionally it is assumed that a city will gain approximately the same number of inhabitants annually, leading to a straight-line increase of population like that shown by line *B*, Fig. 5. This assumption may be warranted in the case of a few large cities with good transportation facilities into suburban towns, which result in many persons doing business in the large city being enumerated as suburban residents. As their business life is spent in the large city, they must be classed for purposes of water supply and sewerage as inhabitants of both places, a fact often overlooked in engineering discussions of population. Commercial districts often require large sewerage capacity, yet the census taker will report a very small population in them. Small cities

without industries, which are the commercial centers of prosperous agricultural regions, occasionally show a tendency toward uniform rate of growth, but if they become producers as well as traders, through the development of manufacturing, they are likely to grow for a time more rapidly than the assumption of arithmetical progression will indicate.



Population at beginning of decade.

FIG. 6.—Relation of rate of growth of population to total population in American cities.

4. *Assumed Decrease in Percentage Rate of Growth.*—As a rule it is found that the larger the city becomes, the smaller will be the percentage rate of growth from year to year. In Fig. 6 there are shown diagrams of the rates of growth of ten of the large cities of this country, in which the percentage growth in 10 years is plotted as ordinate against the population at the beginning of the decade as abcissa. The general reduction in rate of growth as the cities increase in size is distinctly marked.

The curve representing the averages, after smoothing the irregularities, is shown by the heavy line. All of the figures for Chicago and a portion of those for Philadelphia have been omitted as representing conditions so abnormal that they cannot fairly be included, even in making up averages. There is not only a decrease in the percentage rate of growth of cities as they grow in size, but there is also a general decrease in the percentage rate of growth of the entire population of the country, as shown in Table 3. The rate varies with difference in geographical location as indicated in Table 4, which also shows the variations in rate of change in cities and towns of different size and in rural areas. Cities of 25,000 to 100,000 population are growing most rapidly, while the rate of increase in rural population is very much smaller, being only 5.4 per cent in 1910–1920. Table 5 shows the rate of growth in the larger cities by decades since 1840.

TABLE 3.—POPULATION AND RATE OF GROWTH OF UNITED STATES

Date	Population	Growth during decade	
		Numerical	Per cent
1790	3,929,214		
1800	5,308,483	1,379,269	35.1
1810	7,239,881	1,931,398	36.4
1820	9,638,453	2,398,572	33.1
1830	12,866,020	3,227,567	33.5
1840	17,069,453	4,203,433	32.7
1850	23,191,876	6,122,423	35.9
1860	31,443,321	8,251,445	35.6
1870	39,818,449[1]	8,375,128	26.6[1]
1880	50,155,783	10,337,334	26.0
1890	62,947,714	12,791,931	25.5
1900	75,994,575	13,046,861	20.7
1910	91,972,266	15,977,691	21.0
1920	105,710,620	13,738,354	14.9

[1] Census reports claim a deficiency in enumeration of Southern states for 1870. The Census Bureau gives estimated population and percentage as stated. The actual population as returned for 1870 was 38,558,371.

As a rule the assumption of a decreasing percentage rate of increase is the most reliable of the mathematical methods of estimating future populations, particularly if it is checked by basing the prediction on the experience of comparable cities

TABLE 4.—RATE OF GROWTH OF AMERICAN CITIES,[1] BY GEOGRAPHICAL DIVISIONS, 1900–1910 AND 1910–1920

Division	Cities over 100,000 in 1910		Cities over 100,000 in 1920		Cities of 25,000 to 100,000 in 1910		Cities of 25,000 to 100,000 in 1920	
	Number in 1910	Increase 1900–1910, per cent	Number in 1920	Increase 1910–1920, per cent	Number in 1910	Increase 1900–1910, per cent	Number in 1920	Increase 1910–1920, per cent
New England	8	21.2	11	15.5	34	29.0	35	20.2
Middle Atlantic	11	30.8	15	17.6	44	34.0	48	24.3
East North Central	10	32.3	12	36.9	38	37.8	61	43.5
West North Central	5	30.4	7	20.0	17	25.2	14	23.2
South Atlantic	4	20.3	6	33.0	16	37.9	25	43.5
East South Central	4	34.6	4	15.8	7	21.9	7	22.6
West South Central	1	18.1	5	38.6	12	92.2	12	65.2
Mountain	1	51.9	2	22.4	5	54.5	5	16.9
Pacific	6	97.3	6	37.0	6	108.3	12	47.5
United States	50	32.8	68	24.9	179	37.9	219	33.0

Division	Cities of 2,500 to 25,000 in 1910		Cities of 2,500 to 25,000 in 1920		Territory rural in 1910		Territory rural in 1920	
	Number in 1910	Increase 1900–1910, per cent	Number in 1920	Increase 1910–1920, per cent	Number in 1910	Increase 1900–1910, per cent	Number in 1920	Increase 1910–1920, per cent
New England	320	16.7	246	14.7	..	−0.5	..	0.9
Middle Atlantic	444	29.7	541	21.0	..	8.7	..	4.7
East North Central	474	26.0	513	21.6	..	0.0	..	−0.6
West North Central	260	27.5	301	18.1	..	6.0	..	1.7
South Atlantic	190	42.7	242	28.3	..	12.3	..	7.8
East South Central	115	36.1	158	20.5	..	7.5	..	2.4
West South Central	177	80.7	243	35.8	..	27.1	..	8.7
Mountain	91	76.4	110	27.7	..	53.4	..	28.0
Pacific	103	109.3	147	41.4	..	46.4	..	22.6
United States	2,173	36.1	2,500	23.0	..	11.2	..	5.4

[1] Not all places included herein are organized as cities.

which have already passed the present population of the city under consideration. This is done, as shown in Fig. 5, by arranging the lines indicating the change in population of different cities so that when they reach the present population of the city under consideration, they all pass through the same point. This method will tend to give slightly too high percentage rates of increase, since it does not allow for the average progressive decrease throughout the entire country as the years go by.

TABLE 5.—RATE OF GROWTH OF AMERICAN CITIES BY DECADES

Decade	Cities between 100,000 and 200,000		Cities between 200,000 and 400,000	
	Number	Rate of increase, per cent	Number	Rate of increase, per cent
1840–1850	5	50.5		
1850–1860	5	63.2	2	80
1860–1870	6	52.4	6	60
1870–1880	10	49.5	6	26.5
1880–1890	12	76.8	9	40.8
1890–1900	19	33.4	13	25.9
1900–1910	22	38.8	17	42.2
1910–1920	35	27.9	17	27.1

25. Increase in Area.—There is a marked tendency at present, doubtless encouraged by constantly improving transportation facilities, for the inhabitants of cities to move into suburban districts. As the suburban areas become more thickly populated, the improvements of the cities are desired there and are ultimately demanded. To secure these, it often becomes necessary for suburban districts to be annexed to the city, thus extending the city limits. It is reasonable, therefore, to expect a city to increase in area as well as population. In making studies of the future sewerage needs of Fort Wayne, for instance, the authors estimated that the area would grow from 8.6 square miles in 1910 to 17.3 square miles in 1950. In a number of places, municipal boundaries have been ignored in water supply and sewerage undertakings, as at Boston, Mass., and several sections of the territory about New York.

This tendency toward an increase of area by the absorption of suburbs is shown by the growth of Cincinnati since it was incorporated as a city in 1819:

Year	1819	1850	1870	1890	1900	1910	1920
Area, square miles	3.00	6.16	19.05	23.73	35.27	50.26	71.14

Such enlargements of area may cause large and sudden increases in population, which if not anticipated, may result in overtaxing main sewers within the period for which they were intended to be adequate.

Increases in area may also require long extensions of main sewers and may result in greatly increased flow of ground water. Where the community is served by combined sewers, there is also the probability that for considerable periods in the future, or until the population becomes quite dense, brooks will be turned into the main sewers, thus adding materially to the nominal dry-weather flow of sewage. It is also of vital importance to consider where the estimated increase in area will occur in order that the lower sections of the sewer may be placed at elevations from which extensions can be made into new territory which may become populated within the period of design.

26. Density of Population.—The density of population, in persons per acre, in American cities is given in Table 6. The density within a city varies greatly, and it is difficult to estimate its probable future changes. A residential section of the present decade may become a commercial or manufacturing district in the next decade, or the detached homes of persons of means may be replaced by crowded tenements or commercial developments. As estimates of the future population in the different parts of the city are part of the basic data upon which sewerage works are designed, and they must be determined by the exercise of sound judgment, the importance of studying present tendencies toward growth or stagnation, and their causes, is evident.

The difference in policy of cities, with respect to the extension of their boundaries, has been marked. In Eastern communities the city limits have usually been rather more restricted than in Western cities. This tendency has led to rather dense population in the Eastern cities and sparse average population in Western cities of like size. Scattered population involves greatly increased cost of the public utilities or services of various kinds and far greater difficulty of accurate forecast of the probable future direction and degree of development and of increase in population.

Table 6.—Average Density of Population for the 52 Cities of Over 50,000 Population in Which the Density is Greatest
(Computed from data in Census Bureau's "Financial Statistics of Cities" for 1910 and 1921)

City	Density, persons per acre			Population, 1920	Area of land surface within city limits, acres, 1920
	1920	1910	1900		
1. Hoboken, N. J.	82	85	71	68,166	830
2. Somerville, Mass.	37	30	24	93,091	2,518
3. Jersey City, N. J.	36	32	25	298,103	8,320
4. Passaic, N. J.	32	26	13	63,841	2,002
5. New York, N. Y.	30	26	19	5,620,048	191,360
6. Bayonne, N. J.	30	22	13	76,754	2,544
7. Milwaukee, Wis.	28	25	22	457,147	16,290
8. Newark, N. J.	28	23	21	414,524	14,912
9. Boston, Mass.	27	27	23	748,060	27,635
10. Trenton, N. J.	27	22	16	119,289	4,490
11. Cambridge, Mass.	27	26	23	109,694	4,002
12. Paterson, N. J.	26	24	20	135,875	5,157
13. Altoona, Pa.	26	25	23	60,331	2,317
14. Wilmington, Del.	25	22	19	110,168	4,495
15. Norfolk, Va.	24	19	16	115,777	4,800
16. Camden, N. J.	24	21	17	116,309	4,915
17. Charleston, S. C.	24	24	23	67,957	2,874
18. Pittsburgh, Pa.	23	20	13	588,343	25,530
19. Chicago, Ill.	22	19	14	2,701,705	123,383
20. Philadelphia, Pa.	22	19	16	1,823,779	81,920
21. Cleveland, Ohio.	22	19	17	796,841	36,089
22. Lawrence, Mass.	22	21	15	94,270	4,317
23. Wilkes-Barre, Pa.	22	21	16	73,833	3,326
24. Detroit, Mich.	21	18	16	1,044,860	49,839
25. Providence, R. I.	21	19	15	237,595	11,388
26. Lancaster, Pa.	21	19	16	53,150	2,530
27. St. Louis, Mo.	20	17	15	778,897	39,040
28. Buffalo, N. Y.	20	17	14	506,775	24,894
29. Harrisburg, Pa.	20	19	17	75,917	3,766
30. East Orange, N. J.	20	14	9	50,710	2,516
31. San Francisco, Calif.	19	14	12	506,676	26,880
32. Savannah, Ga.	19	16	18	83,252	4,473
33. Johnstown, Pa.	19	20	15	67,327	3,488
34. Chester, Pa.	19	13	11	58,030	3,020
35. Reading, Pa.	18	24	20	107,784	6,091
36. Schenectady, N. Y.	18	15	11	88,723	5,019
37. Portsmouth, Va.	17	21	19	54,387	3,136
38. Columbus, Ohio.	16	14	12	237,031	14,427
39. Louisville, Ky.	16	17	16	234,891	14,349
40. Dayton, Ohio.	15	12	13	152,559	10,107
41. Bridgeport, Conn.	15	13	9	143,555	9,370
42. Lynn, Mass.	15	13	10	99,148	6,705
43. Elizabeth, N. J.	15	12	9	95,783	6,191
44. Evansville, Ind.	15	16	14	85,264	5,577
45. Racine, Wis.	15	13	10	58,593	3,858
46. Covington, Ky.	15	17	24	57,121	3,837
47. Baltimore, Md.	14	29	26	733,826	50,560
48. Rochester, N. Y.	14	17	14	295,750	20,566
49. Syracuse, N. Y.	14	12	10	171,717	11,849
50. New Haven, Conn.	14	12	9	162,537	11,460
51. Hartford, Conn.	14	9	7	138,036	10,162
52. Tulsa, Okla.	14	..	..	72,075	5,002

Note.—Some cities show a decrease in density between censuses, due to the annexation of large areas of adjacent territory.

27. Effect of Zoning.—"The most direct effect that the engineer realizes in undertaking to design a sewerage system in a community which has been zoned and which has an effectively administered zoning ordinance, is the relative certainty and economy with which he can adapt the sewers to the service they will be called on to perform . . . The perfection of sewer performance must be proportioned to property values. In zoned areas the property values are relatively easy to determine and also relatively stable, so that the engineer has a much more reliable guide to judgment than he has when the future character and, therefore, the future value of the property is uncertain."[1]

Hansen has shown that, with a typical block 330 by 660 ft. and lots 50 ft. wide, the densities of population and percentages of impervious area would be about as follows:

Character of district	Development	Density of population, persons per acre	Impervious surface, per cent
Dense residential	Two-family houses and six-family apartment buildings	55	60
Medium residential	Mostly single-family houses	35	43
Light residential	Single houses only, some on double lots	15	31
Mercantile	....	14	100
Light commercial	....	30	80
Industrial	....	10	60

Zoning may also be advantageous in preventing low-lying land from being developed in such a way as to require unreasonably expensive drainage or sewerage, or at least to control the uses of such districts so as to avoid the worst difficulties attendant upon poor natural drainage.

28. Accuracy of Population Estimates.—It appears that in the majority of cases, forecasts of population made by the methods described above indicate somewhat larger populations than those actually found by census counts. From a comparison

[1] HANSEN, PAUL: The Relation of Zoning to the Design of Drainage and Sewerage Systems. *Trans.*, Am. Soc. C. E., 1925; **88**, 680.

of some 40 population estimates with census figures, a number of them covering periods of 35 years since the date of the census preceding the estimate, it appears that on the average the estimated population has exceeded the actual population by about 1 per cent for each year which has elapsed since that census.

As a rule, curves of actual population growth are not regular but fluctuate somewhat widely. Curves of predicted population are always smooth curves. Therefore, it is not to be expected that actual populations for any particular place should correspond closely to the estimated population for each of several census dates. The deviation is likely to be more marked with smaller than with larger cities since the former are more susceptible to various conditions and factors affecting the rate of growth. Therefore, it is possible that an estimate which may ultimately prove to be a good representation of average growth may depart widely from each of several census figures. Differences amounting to 20 per cent or even more do not necessarily indicate that the general trend is diverging materially from the estimate.

29. Proportion of Municipal Water Supply Reaching Sewers.—Since much of the sewage consists of spent water from the water works supplying the city, the portion of the water supply which will reach the sewers must be estimated. A considerable part of the water used by railroads, by manufacturing establishments and power plants, for street and lawn sprinkling, for extinguishing fires, and by consumers not connected with sewers, does not reach the sewers. There is also some leakage from the water mains and service pipes which does not reach the sewers. In 1911 the Milwaukee Sewage Disposal Commission estimated that of the total daily supply of 105 gal. per capita 40 gal., or 38 per cent of the supply, never reached the sewers. This 40 gal. included:

	Gal. per capita daily
Steam railroads	5
Industrial uses	5
Street sprinkling	5
Lawn sprinkling	2½
Consumers not connected with sewers	7½
Leakage from water mains and services	15
Total	40

In a few cities where the infiltration of ground water and the discharge of roof water into sewers is low, the quantity of sewage has been actually less than the quantity of water supplied, as at Brockton, Mass., where the sewage has ranged from 56 to 114 per cent of the water supply. Only 3 out of the 27 years considered showed 100 per cent or greater.

Often, however, infiltration, roof water, and water used in industries and obtained from privately owned sources, make the quantity of sewage larger than that of the public water supply. This is shown by the ratio of sewage to water supply in the following places, between 1900 and 1926:

In general, the total quantity of sewage is somewhat greater than the total water consumption.

Locality	Percentage of Sewage to Water
Massachusetts North Metropolitan Sewerage District	111–145
Worcester, Mass.	109–158
Quincy, Mass.	105–143
Providence, R. I.	102–195

With well-built sewers and with roof water excluded, the variation from year to year in the ratio of sewage to water supply in a city is not great, unless there is a substantial change in the industrial uses of water. For this reason, the consumption of water in a city affords basic data for sewerage as well as water-supply engineering.

30. Water Consumption in Cities.—The consumption of water in cities is usually expressed in gallons per capita daily. A direct comparison of such records from different cities is likely to be misleading because in some cities large quantities of water used industrially are obtained from privately owned supplies, as at Fall River, and in other cities, as Philadelphia and Buffalo, the industries use the municipal supply mainly. Furthermore, the care taken to reduce the waste of water through leaks in mains, services and plumbing has a decided effect on the per capita consumption. Metering the individual consumers' supplies indirectly prevents waste of water from plumbing and by users and tends to keep the consumption reasonably low. The waste and unaccounted-for water in metered systems may range in practice from 20 to 40 per cent, more or less, of the total water entering the supply-pipe system, and very much more in unmetered systems. In any particular case, therefore, it is important to ascertain the present practice in preventing waste of water, the probability that more attention will be paid

to such work, and the probable development of large industrial uses. In such studies valuable aid can be obtained from a report[1] on water consumption by Metcalf, Gifford and Sullivan. Some data on consumption in typical American cities are given in Table 7.

TABLE 7.—WATER CONSUMPTION AND METERING IN TYPICAL AMERICAN CITIES

Place	Item	1900	1905	1910	1915	1920	1925
Brockton, Mass.	Population	....	55,700	66,000	74,588	79,729	83,548
	Consumption	....	36	36	38	43	50
	Per cent metered	....	90	99	100	100	98
New Bedford, Mass.	Population	62,500	75,000	96,652	110,000	131,350	146,800
	Consumption	101	95	79	70	78	65
	Per cent metered	15	23	48	96	96	93
Worcester, Mass.	Population	118,421	132,550	150,738	169,599	179,923	191,000
	Consumption	69	73	71	75	90	82
	Per cent metered	94	96	97	100	97	97
Providence, R. I.	Population	187,297	214,335	246,000	278,727	270,472	312,615
	Consumption	54	68	63	62	80	75
	Per cent metered	83	86	89	93	94	97
Hartford, Conn.	Population	....	91,000	105,500	138,000	170,000	188,000
	Consumption	....	69	70	67	71	86
	Per cent metered	....	99	99	98	97	96
Cleveland, Ohio	Population	397,200	462,000	604,073	765,000	925,283	1,098,466
	Consumption	169	131	102	102	152	148
	Per cent metered	6	70	98	99	98	99
Cincinnati, Ohio	Population	325,902	343,254	363,591	410,000	401,247	416,000
	Consumption	117	129	128	119	123	119
	Per cent metered	8	12	33	65	100	100
Indianapolis, Ind.	Population	....	....	237,000	269,500	322,200	372,000
	Consumption	....	....	81	82	94	93
	Per cent metered	....	....	11	12	14	31
Milwaukee, Wis.	Population	285,315	335,000	390,000	450,000	500,000	565,000
	Consumption	83	91	109	105	133	128
	Per cent metered	86	94	98	99	99	99
New Orleans, La.	Population	....	....	....	363,100	387,219	411,000
	Consumption	....	....	....	71	107	120
	Per cent metered	....	....	....	100	100	100
San Antonio, Tex.	Population	....	....	99,000	132,000	161,000	
	Consumption	....	....	128	117	126	
	Per cent metered	....	....	18	31	43	
San Francisco, Cal.	Population	343,000	384,000	417,000	500,000	575,000	
	Consumption	74	91	85	85	65	
	Per cent metered	23	21	27	31	100	

Population served by water works.
Consumption in gallons per day per capita.
Percentage of services which are metered.

From the statistics available two conclusions seem warranted: First, there is a tendency toward a gradual increase in the quantity of water used per capita of population. This is

[1] *Jour.*, N. E. Water Works Assoc., 1913; **27, 29.**

undoubtedly due, so far as it relates to domestic uses, to improvements in plumbing and an increasing appreciation of general cleanliness. The number of fixtures per person, as well as the quantity of water required per fixture, has greatly increased in recent years. In the larger cities, the increased consumption may be due in part to the difficulties surrounding the management of the water departments, which are usually much greater than in the smaller cities and towns. Second, the evidence furnished by such cities as Providence, Worcester, Fall River and Lawrence, indicates that with careful management, aided, perhaps, by thorough metering, it is possible to hold down the increase in quantity of water consumed to reasonable proportions.[1]

31. Rate of Water Consumption in Different Parts of a City.—The rate of water consumption per capita varies greatly in different parts of a city. Investigations in 19 cities made in 1910 and 1911 showed[2] that the range was as follows:

Class of buildings	Water consumption in gal. per day per capita	
	Range	Average
Apartment houses	24–135	62
First-class dwellings	15– 75	54
Middle-class dwellings	11– 66	34
Lowest-class dwellings	4– 37	15

The industrial uses vary greatly, according to the nature of the manufacturing. Fuertes reported[3] in 1906 that the range of such use in 11 cities was from 0.4 gal. per capita daily in Wellesley, Mass., to 81 gal. in Harrisburg, Pa. It was 30 gal. in Boston, 40 gal. in Cleveland and 45 gal. in Milwaukee. These figures are based on the entire populations of the cities, but as manufacturing is carried on only in certain districts, as a rule, the per capita consumption for manufacturing in those districts is much higher than the average for the city. In practical designing work it is desirable to make an actual inspection of the industries and a careful estimate of the quan-

[1] See also METCALF, LEONARD: Effect of Water Rates and Growth in Population upon Per Capita Consumption. *Jour.*, Am. W. W. Assoc., 1926; **15**, 1.

[2] *Jour.*, N. E. Water Works Assoc., 1913; **27**, 29.

[3] "Waste of Water in New York."

tities of water they use from all sources, and the wastes they produce. The same is true of the consumption in business districts.

32. Fluctuations in Water Consumption.—While it is important to know the average quantity of water consumption, it is of still greater value to have data relating to the fluctuations, as a sewer must be designed to take the sewage when flowing at its maximum rate. The maximum rate of water consumption usually occurs during summer months when water is in demand for street and lawn sprinkling and the excess is not likely to reach the sewers, or in the winter when large quantities are allowed to run to prevent freezing of pipes and fixtures, this excess usually finding its way into the sewers. Records of maximum water consumption for 67 Massachusetts cities and towns (1910) have been compiled by a Committee on Water Consumption Statistics and Records.[1] The average figures are as follows:

Average water consumption, gallons per capita per day...	63
Daily average in maximum month, percentage of average for year...	128
Daily average in maximum week, percentage of average for year...	147
Maximum consumption in one day, percentage of average for year...	198

There were, however, instances in which the maximum rates greatly exceeded these averages. For example, in Manchester and Mansfield, Mass., the maximum daily consumption was 302 and 368 per cent of the average for the year, respectively. These high rates of consumption, however, almost always occur at times when the usual proportion of the flow does not reach the sewers, as in the driest portion of the summer, or in winter when water from other sources, as for example, ground water, is likely to be at a minimum.

In addition to the fluctuations in flow already discussed, there is an important variation from hour to hour each day, as illustrated by Fig. 7, which shows the relationship of sewage flow to water delivered at Denver, Colo.

As is seen from the figure, the sewage discharge curve closely parallels the water consumption curve but with a lag of several hours. The average sewage flow for the month of 46.2 m.g.d.,

[1] *Jour.*, N. E. Water Works Assoc., 1913; **27**, 29.

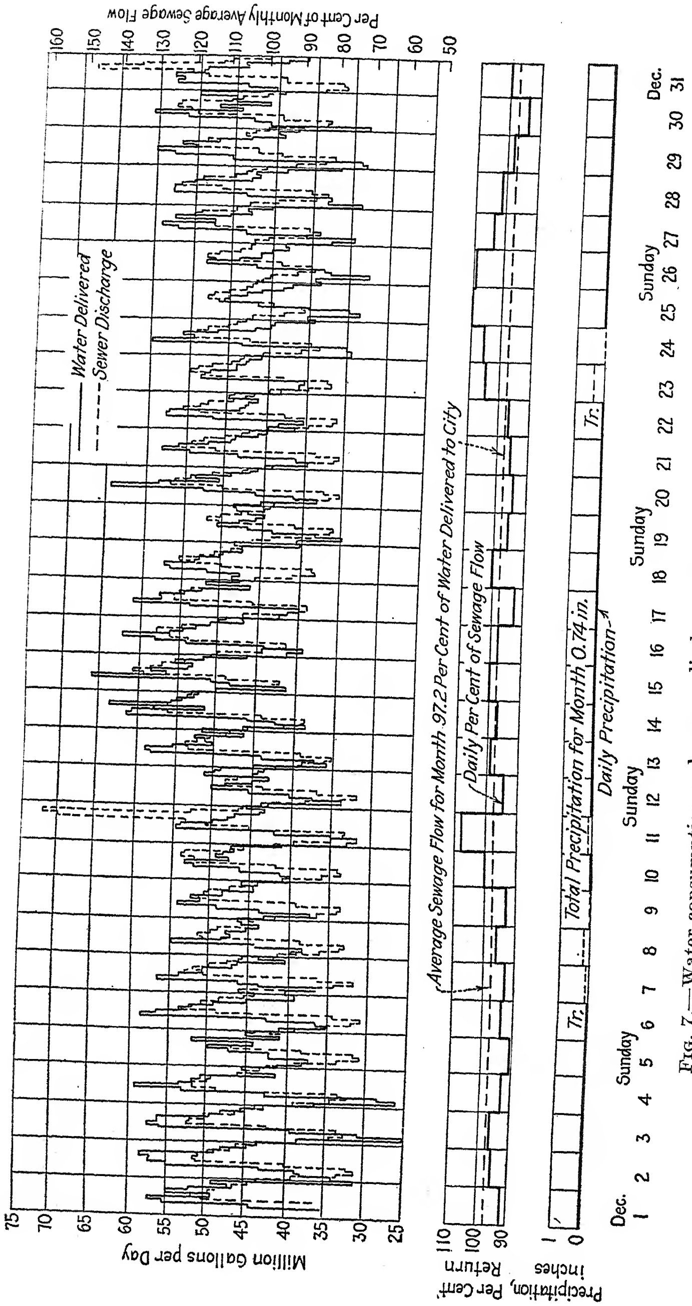


FIG. 7.—Water consumption and sewage discharge at Denver, Colo. for December 1926.[1] The sewage discharge curve lags behind the water supply curve from two to four hours.

[1] *Eng. News Rec.* 1928; **100**, 556.

or 136 gal. per capita daily, was 97.2 per cent of the water consumption, which was 47.6 m.g.d. The maximum rate of sewage flow for a 2-hr. period reached 156.8 per cent of the monthly average on Dec. 11, followed by 148.3 per cent on Dec. 31 and 131.3 per cent on Dec. 15. The maximum was due in part to the effect of rainfall, shown in a sudden increase in discharge for a few hours' duration.

The hourly fluctuation in rate of water consumption has a decided effect upon the rate of sewage flow, as discussed later in this chapter. It is not, however, entirely responsible for the fluctuation in the rate of flow of sewage, for in some places large quantities of water used by industrial establishments and obtained from other sources than the public supply are discharged into the sewers during the working hours of the day, thus tending to increase the peak flow beyond the amount resultting from the normal fluctuation in the draft on the municipal water supply.

In the absence of more authoritative information, an additional allowance of 50 per cent over the average 24-hour rate of water consumed, may be made for the excess of the hourly peak over the average 24-hour rate. The figure, however, will vary with the locality. If this peak consumption is applied to the maximum draft for a single day, of 198 per cent of the yearly average, and it is assumed that the portion of the water supply which finds its way into the sewers averages 50 gal. per capita per day, there will be maximum rate of contribution from the public and private water supplies of about 150 gal. per capita daily ($50 \times 1.98 \times 1.50 = 148.5$). This will serve to illustrate the method of estimating the flow of sewage from the water consumption, but should only be applied in design when local conditions are found to warrant it. Some engineers use a quantity, such as 200 gal. per capita daily, as the maximum rate of contribution originating from water supplies, taking a quantity which their experience has indicated as adequate and yet not in excess of flows which have been observed in existing sewers. Data from actual sewer gagings are, of course, much to be preferred as a basis of design where these are available. At Cincinnati (see pp. 59 to 62) such gagings were made and data derived from these furnished the basis for the design.

33. Ground Water.—A portion of the rainfall ordinarily runs quickly into the storm-water drains or other drainage

channels. Another portion is evaporated or is absorbed by vegetation, and the remainder percolates into the ground, becoming ground water. The proportion which thus percolates into the ground depends upon the character of the surface and soil formation and upon the rate and distribution of the precipitation according to seasons. Any reduction in permeability, as, for instance, that due to buildings, pavements, or frost, decreases the opportunity for precipitation to become ground water, and increases the surface run-off correspondingly. The amount of ground water flowing from a given area may vary from a negligible amount for a highly impervious district or a district with a dense subsoil, to 25 or 30 per cent of the rainfall for a semi-pervious district with a sandy subsoil permitting rapid passage of water into it. The percolation of water through the ground from rivers or other bodies of water sometimes has considerable effect on the groundwater table[1] which rises and falls continually.

The presence of ground water results in leakage into the sewers and an increase in the quantity of sewage and the expense of disposing of it. This infiltration from water-bearing earth may be as low as 5,000 gal. per day per mile of sewers and may be as high as 40,000 gal. or more. During heavy rains, when there may be leakage through manhole covers, as well as infiltration into the sewers themselves, the quantity may exceed 100,000 gal. per day per mile of sewer. It is a variable part of the sewage, depending upon the quality of the materials and workmanship in the sewers and building connections, the character of the maintenance and the height of the ground-water table.

The sewers first built in a district usually follow the water courses in the bottoms of valleys, close to and occasionally in the beds of brooks. As a result, these old sewers may receive comparatively large quantities of ground water, while sewers built later at higher elevations in the districts will receive relatively smaller quantities of ground water. With an increase in the percentage of area in a district which is paved or built over, comes an increase in the percentage of storm water which is conducted rapidly to the drains and water courses, and a decrease in the percentage of the storm water which can percolate into the earth and tend to leak into the sewers. A sharp distinction is to be made between maximum and average rates of leakage or

[1] The ground water table is the surface of the water in the soil.

TABLE 8.—INFILTRATION INTO SEWERS AT VARIOUS PLACES

Place	Sizes	Miles of sewer	Miles below ground water	Infiltration		Kind of joints	Remarks
				Gallons per day per mile	Gallons per day per mile below ground water		
Long Beach, Calif.	8.10	5.67	2.73	12,600	26,200	Cement	Tested before house connections were made
Long Beach, Calif.	8.18	12.41	3.38	1,780	7,450		
Long Beach, Calif.	8.24	13.60	1.71	690	5,490		
San Luis Obispo, Calif.	....	.....	....	30,000			
Santa Cruz, Calif.	....	.....	....	......	2,110[1]	....	4 gal. per day per inch diameter per 100 ft.
Monte Vista, Col.	....	8	....	650,000	....	Cement	80 per cent of total flow
New Haven, Conn.	....	.....	....	18,000			
St. Petersburg, Fla.	....	.....	....	13,000			
Kewanee, Ill.	....	.....	....	10,000–15,000			
Detroit, Mich.	....	.....	....	......	....	....	1,000 gal. per acre daily
Billings, Mont.	....	.....	....	......	....	....	1,500 gal. per acre daily
East Orange, N. J.	....	.....	....	27,000			
Rutherford, N. J.	....	.....	....	10,000			
East Aurora, N. Y.	....	.....	....	2,000			
Goldsboro, N. C.	....	.....	....	......	....	....	⅓ capacity of sewer
Akron, Ohio	.....	.....	....	1,000			
Salem, Ohio	....	.....	....	12,000			
Altoona, Pa.	8.12	2.8	....	2,140	....	Compounds	
Charleston, S. C.	....	.....	....	46,000 to 18,000			
Jacksonville, Tex.	...	.....	....	......	....	....	50 per cent of flow
Salt Lake City, Utah	....	.....	....	......	40,000±	....	0.061 cu. ft. per second per mile, before connections
Seattle, Wash.	....	.....	....	......	....	....	1 gal. per minute per acre
Kitchener, Ont.	....	.....	....	10,000			

This table has been prepared from data compiled from the answers to a questionnaire in "Public Works," 1927; **58**, 363.
[1] Assuming that average diameter is 10 in.

infiltration into sewer systems. The former are necessary in determining required sewer capacities: the latter, in the discussion of operating problems such as the annual cost of pumping sewage. The records of measured leakage of ground water into sewers given in Tables 8 to 11, inclusive, are significant. In Table 8, the amount of infiltration occurring in various cities

Table 9.—Leakage in North Metropolitan Sewer District, Boston, in April and May

	Average	Maximum	Minimum
Gallons per capita per day	62.2	93.8	38.7
Gallons per acre per day	1,738	2,577	1,094
Gallons per mile of sewer per day	50,600	78,900	30,900

Table 10.—Estimate of Infiltration into Sewers at Worcester, Mass., 1890–1897
(Million gallons per day)

Month	1890	1891	1892	1893	1894	1895	1896	1897
January	.....	1.508	1.610	1.719	1.834	1.966	2.056	2.139
February	.....	1.014	1.083	1.156	1.234	1.323	1.383	1.439
March	.....	1.275	1.361	1.453	1.550	1.663	1.738	1.808
April	.....	2.701	2.885	3.080	3.286	3.524	3.684	3.832
May	.....	1.628	1.738	1.856	1.980	2.122	2.220	2.309
June	1.254	1.350	1.442	1.539	1.643	1.761	1.840	1.915
July	1.003	1.080	1.153	1.231	1.314	1.409	1.473	1.532
August	0.964	1.038	1.109	1.183	1.263	1.354	1.416	1.472
September	1.325	1.427	1.524	1.628	1.736	1.861	1.946	2.024
October	1.334	1.435	1.533	1.637	1.747	1.873	1.958	2.037
November	0.906	1.975	1.042	1.112	1.187	1.272	1.330	1.384
December	1.089	1.172	1.252	1.338	1.427	1.530	1.600	1.663
Averages	1.466	1.466	1.478	1.578	1.685	1.805	1.888	1.963
Total mileage		.63.90	68.22	72.78	78.03	81.60	84.87	88.96
Average gallons per mile per day		22,960	21,650	21,690	21,580	22,120	22,250	22,080
Population on line of sewers		75,644	79,015	83,272	83,588	89,195	90,754	93,737
Gallons per capita per day		19.4	18.7	18.9	20.3	20.2	20.8	20.9

From 1891 to 1897, inclusive, the average per day was 22,047 gal. per mile or 19.9 gal. per capita.

is given, while Table 9 gives the leakage in the North Metropolitan Sewer District, Boston, in the three units most frequently used for this purpose. In Table 10, the monthly variations in infiltration over 7½ years at Worcester, Mass. are shown. The leakage varies considerably from month to month, being at a maximum in April and May; while the yearly averages increased 33.8 per cent during the period under consideration. The data in Table 11 are from weir measurements of flow of sewers discharging into Ten Mile Creek, made during dry weather.

TABLE 11.—ACTUAL FLOW OF DOMESTIC AND INDUSTRIAL SEWAGE AND GROUND WATER AT TOLEDO, OHIO, 1917[1]

Sewer district	27	26	16	22	Totals and means
Population, total	2,634	3,902	17,087	17,838	41,461
Connected to sewers	1,889	3,673	16,842	16,308	38,712
Area, acres	280	288	1,083	502	2,153
Length of sewers, miles	6.73	5.96	17.82	11.70	42.21
Domestic sewage:[2]					
Gallons per day per capita					
of total population	20	19	34	10	21
of population connected to sewers	28	20	34	11	23
Industrial sewage:[2]					
Gallons per day per acre	4,900	4,000	20,000	127,000	14,000
Ground water:[3]					
Gallons per day per acre	557	736	1,100	246	783
Gallons per day per mile of sewer	23,400	35,600	67,000	10,000	40,000

[1] *Eng. News-Rec.*, 1918; **80**, 1233.
[2] Water supply metered to users.
[3] Minimum night flow less simultaneous industrial sewage.

The quantity of infiltration depends on the length of sewer, the area served, the soil and topographic conditions and to a certain extent on the population, which affects the number and total length of house connections. The leakage through defective joints, porous concrete and cracks is large enough in many cases to lower the ground-water table to the crown of the sewer and frequently well down toward the invert, although the elevation of the water table varies with the quantity of rain and snow water percolating into the ground.

34. Daily Flow of Sewage.—In Table 12 are given average statistics of the sewage of a number of Massachusetts towns

TABLE 12.—MAXIMUM AND AVERAGE FLOWS OF SEWAGE IN MASSACHUSETTS CITIES, 1920[1]

Place	Population census of 1920	Average yearly quantity of sewage, gallons per 24 hr.			Average quantity of sewage in maximum month, gallons per 24 hr.		
		Per inhabitant	Per connection	Per mile of sewer	Per inhabitant	Per connection	Per mile of sewer
Andover	8,268	30	330	......	42	460	
Attleboro	19,731	30	510	19,000			
Brockton	66,254	42	410	31,500			
Clinton	12,979	84	690	50,800			
Concord[5]	6,461[4]	82	1,120	59,800	112	1,520	81,400
Fitchburg	41,029[3]	73	....	52,400	102	....	73,200
Framingham[5]	17,033	74	540	47,700	124	910	79,900
Franklin[5]	6,497	32	390	13,100			
Gardner	16,971	59	550	35,5.0			
Hopedale	2,777	31			57⊙		
Hudson	7,607	58	650	42,400	83⊙	930⊙	61,200⊙
Marlborough	15,028	72	500	36,100	121	840	61,000
Milford	13,471	72	740	54,200	75	770	56,300
Natick	10,907	137	1,060	......	359	2,780	
North Attleborough	9,238	84	1,200	46,800	92⊙	1,320⊙	51,200⊙
Northbridge[5]	10,174	57					
Norwood	12,627	95	1,060	67,300			
Pittsfield	41,763	84	730	57,500			
Westborough[5]	5,789	91	950	55,100	153	1,590	92,400
Worcester[2]	179,754	135	.....	131,000			

[1] From Annual Report of the Massachusetts Department of Health, (1920).
[2] Includes flow from 69.81 miles of combined sewers.
[3] Largely combined system.
[4] Approximately two-thirds of the sewers laid below ground water level.
[5] Substantial portion of population not served by sewers.
⊙ Accuracy of these figures is questionable.

and small cities having separate sewers. The gagings of the sewage from several districts in Chicago, Table 13, show rates which exceed greatly those found in Massachusetts. The Chicago sewers are built on the combined system, but the gagings were made in dry weather. The large flow is due mainly to the heavy consumption of water in the city, which averaged 242 gal. per capita daily in 1910.

TABLE 13.—TYPICAL DRAINAGE AREAS AND DRY WEATHER RUN-OFFS, CHICAGO, 1910 AND 1911

(From Wisner's Report on Sewage Disposal, San. Dist. of Chicago, 1911)

Sewer outfalls	Drainage area in acres	Population estimated 1911	Dry weather run-offs,				Density of population per acre	Period covered by observation
			Cu. ft. per sec.	Cu. ft. per sec. per acre	Cu. ft. per sec., per sq. mile	Gal. per cap. per 24 hours.		
Diversey Boulevard (W).	890	23,550	8.65	0.0097	6.22	238	26.4	Aug. 15–17, 1911, 2 days.
Randolph St. (W).	240	11,368	6.10	0.0254	16.25	348	47.4	Aug., 3 days.
Robey St. (S).....	2,500	38,728	10.1	0.0040	2.58	169	15.5	June 1–3, 1911, 2 days.
Ashland Ave., (S)..	980	44,581	23.2[1]	0.0237	15.1	338	45.5	May 18–20, 1911, 2 days.
Center Ave., (S)...	660	23,463	20.9[2]	0.0317	20.3	578	35.6	May 16–18, 1911, 2 days.
Thirty-ninth St., pumping station.	14,340	285,900	140.0[3] 100.0[4]	0.0098 0.0070	6.25 4.46	318 227	20.0	....
Ninety-second St...	98	3,666	1.84	0.0188	12.0	325	37.4	209 days. Aug. 1, 1910–Mar. 31, 1910.
Wentworth Ave., (S), (Calumet).	5,300	30,464	12.4	0.0023	1.5[5]	264	5.8	Aug. 1, 1910–July 31, 1911. 253 days.

[1] Daily variation average
8 A.M. to 8 P.M. 28.5 c.f.p.s. contains large amount of industrial waste.
8 P.M. to 8 A.M. 18.6 c.f.p.s.

[2] Daily variation average
8 A.M. to 8 P.M. 25.7 c.f.p.s. contains large amount of industrial waste.
8 P.M. to 8 A.M. 17.0 c.f.p.s.

[3] This run-off or more for 76 days in 1909.

[4] This run-off or more for 276 days in 1909.

[5] 2.4 c.f.p.s. per square mile occurred 329 days in the year.

The average rates of flow upon which estimates of cost of pumping and treatment may be based are much below the maximum rates, and, from the data available, appear to range in a general way between 100 and 125 gal. per capita per day for the larger cities. For small towns average rates appear to range from 25 to 60 gal. per capita per day.

35. Hourly Flow of Sewage.—The flow of sewage per hour is more important to the designing engineer than the flow per day, because he has to provide sufficient sewer capacity for the largest rate of sewage flow which it is reasonable to expect.

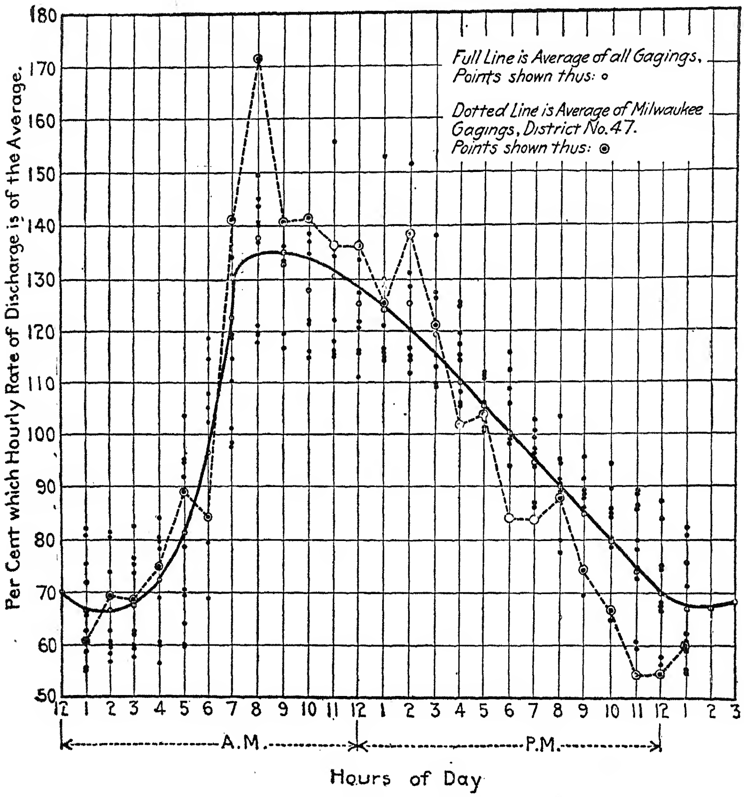


FIG. 8.—Hourly variation in flow of sewage in various cities.

The records used in preparing this diagram were from the following sources: Birmingham, England, average of two years, 1906–7; East Orange, N. J., March 16–17, 1910; Gloversville, N. Y., Oct. 30, 1906, and Sept. 12, 1907; City A, of 15,000 population, typical average curve; Milwaukee, Wis., Oct. 24–28, 1920; Toronto, Ont., 1900 and 1908; Worcester, Mass., Nov. 13, 1909, and March 21–27, 1910.

In Fig. 8 are plotted the hourly rates of flow in several cities, expressed as percentages of the daily average. The smooth curve is drawn through points obtained by averaging points taken from curves representing the hourly rates of flow in the cities named in the note below the illustration. In preparing curves of this nature it is desirable to synchronize them by making

an allowance for the time required for the sewage to flow from the city to the gaging point; if this allowance is not made the data from different cities are not easily compared. The dotted line in Fig. 8 represents the flow of sewage from a large Milwaukee residential district.

36. Sewage from Residential Districts.—The quantity of sewage from a residential district will depend upon its area, density of population, per capita water consumption and the quantity of ground water leaking into the sewers. The future character of such a district should be considered carefully, keeping in mind that there may be a lowering of the per capita

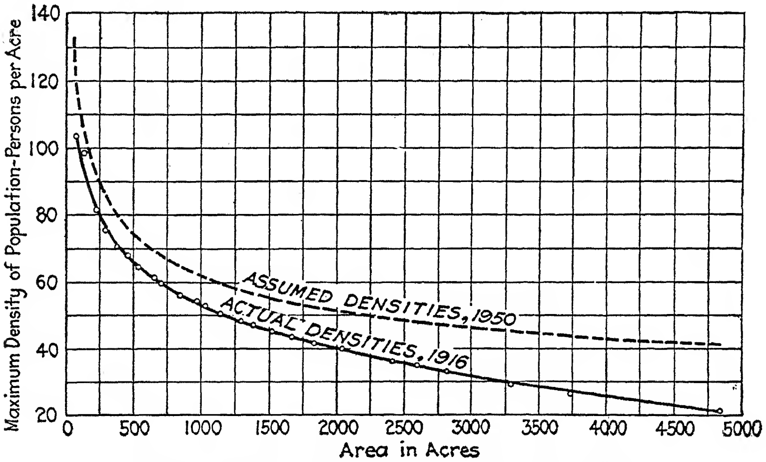


FIG. 9.—Maximum density of population, Lynn, Mass.

consumption of water if widely separated homes of wealthy families give way to more numerous and modest homes or to tenements. In making estimates of the quantity of sewage from such a district, it is incorrect to use the average density of population of the entire city, which rarely exceeds 25 persons per acre, because there are many residential districts where it is much higher. For example, it was found in Philadelphia in 1910, that the population per acre was 58 to 64, where the prevailing building type was pairs of two- and three-story houses, and 97 to 123 where the buildings were solid rows of houses. The sewage from districts of the first class amounted to 9,800 to 14,200 gal. per acre daily, and from those of the second, to 10,500 to 26,300 gal.

The decrease in average density of population with increase in area is a factor of importance in the design of separate and intercepting sewers. It is impossible to predict with certainty the locations of future areas of high density. This is a further reason for liberality in the design of the laterals. Fig. 9 shows the actual maximum densities for different areas found in Lynn in 1916 and the densities estimated for the year 1950. Data sufficiently accurate for making and checking such curves may be obtained by actual count, by the use of assessors' estimates,

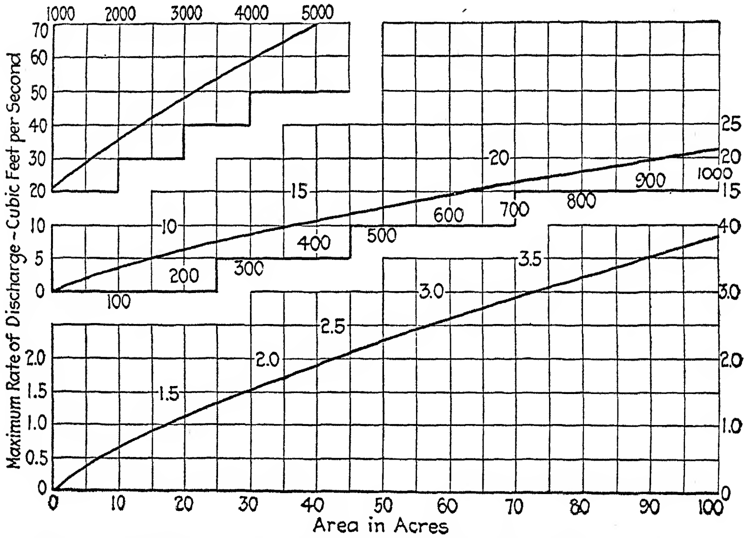
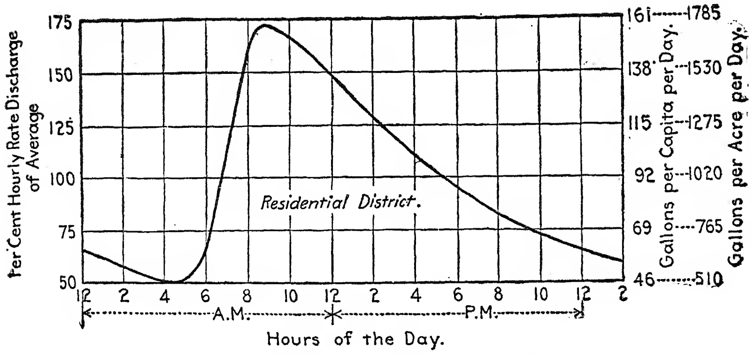


FIG. 10.—Maximum rate of discharge of residential sewage, Lynn, Mass.

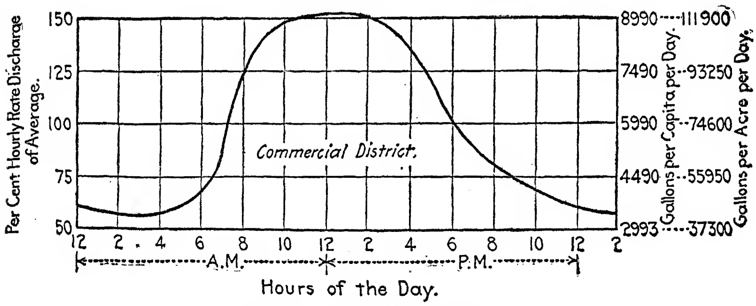
from the census reports where the population is sufficiently large so that it is given by wards and precincts, or from data contained in directories, post office counts and lists of polls.

It is evident from the foregoing that the maximum rate of discharge of residential sewage per unit of area may be much higher for small areas than for larger ones. Fig. 10 shows the estimated maximum rate of discharge of sewage for different areas based upon densities of population shown in the higher curve of Fig. 9 and a maximum per capita rate of sewage flow of 220 gal. per day, otherwise determined for the district under consideration.

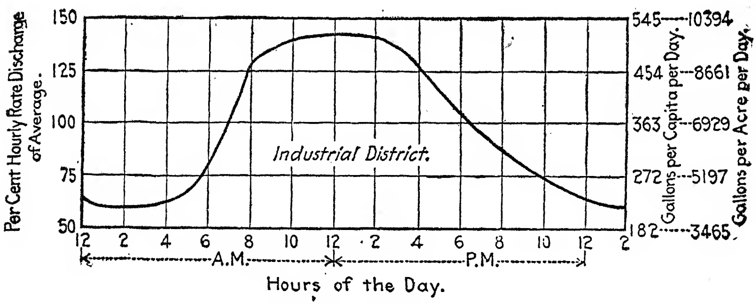


Hourly Variation in Flow of Sewage.

Curve A. Ross (Bloody) Run Sewer.



Curve B. Vine Street Sewer.



Curve C. Marshall Ave. Sewer.

FIG. 11.—Hourly variations in flow of sewage; Cincinnati.

The rate per hour fluctuates greatly in the case of moderately developed medium and high-class residential districts, because of the great variation in the hourly consumption of water. Fig. 11, curve *A*, gives the average hourly rate determined from two sets of gagings of a Cincinnati district of 1,617 acres with a population of 11.1 persons per acre. This district is largely residential, and the maximum observed hourly flow was 273 per cent of the average, although the district is so large that such a wide range in the rate of flow in its main sewer is unusual. In the usual case of a district of this size, the maximum hourly rate may be expected to reach 160 to 170 per cent of the average.

37. Sewage from Commercial Districts.—A commercial district yields much more sewage per acre than a residential district, because of the concentration there during the day of many persons in stores, offices and public buildings, hotels and often some apartment houses. It should be noted that much of this population may be included again in designing for suburban residential areas, so that per capita data for different types of districts yield more useful design data than those for the entire city. A commercial district is relatively small compared with the other districts in a city, but its growth should be estimated carefully on account of the large quantity of sewage coming from it.

The allowance for used water from a mercantile district is more difficult to estimate than that from a residential district. If the estimate is made in connection with the design of intercepting sewers, pumping stations or treatment works for use with existing main sewers, the latter should be gaged and a suitable allowance made for future development. It may also be helpful to ascertain the water supply of the district from both municipal and private sources.

The average quantity of sewage from a 123-acre commercial district in Philadelphia was 92,800 gal. per acre per day. This district was entirely built over, the area occupied by buildings being 80 acres and that by streets 32 acres. This is a rather large quantity of sewage from a commercial district. Gagings of the flow from eight such districts in Cincinnati gave figures ranging from 14,000 to 72,000 gal. with an average of 40,000 gal. per acre daily, as given in Table 14.

The hourly rate of flow from districts of this character varies greatly, as is to be expected, but the high rate of discharge

lasts much longer than does the early morning peak in the curve of flow from residential districts. Curve *B* in Fig. 11 illustrates this, and shows the hourly rate in a district of 23.6 acres with 12.5 persons per acre, the center of business in Cincinnati. The average daily flow in this case was 72,000 gal. per acre and the maximum rate during the gaging period was 139,000 gal. per acre daily.

TABLE 14.—AVERAGE FLOW OF SEWAGE FROM COMMERCIAL DISTRICTS, CINCINNATI, OHIO, 1912

Sewer district	Area in acres	Population		Sewage flow from actual gagings				No. of gagings covering 24-hour day	Dates of gagings
				Gals. per acre per day		Gals. per capita per day			
		Total	Density	Average	Maximum[1]	Average	Maximum[1]		
Sycamore St.	27.8	1,702	61.2	25,800	76,300	421	1,245	1	Oct. 30, 31.
Main St......	18.5	487	26.3	37,750	88,100	1,435	3,350	2	Oct. 29, 30.
Walnut St....	29.2	380	13.0	60,000	135,000	4,610	10,360	1	Nov. 2.
Vine St.......	23.6	294	12.5	72,000	139,000	5,780	11,160	1	Nov. 5
Race St......	37.7	655	17.4	48,250	89,400	2,777	5,150	2	Nov. 15, 16
Elm St.......	32.5	1,226	37.7	40,800	81,250	1,080	2,150	2	Nov. 8, 9
Plum St......	29.5	1,231	41.7	14,700	35,300	352	845	1	Nov. 12
Central Ave.	28.4	1,579	55.62	22,050	38,400	396	690	2	Nov. 12, 13
Totals or averages....	227.2	7,554	33.2	40,169	85,344	2,106	4,369		

[1] Maximum during gaging period.

38. Sewage from Industrial Districts.—The quantity of industrial wastes not originating from the public water supply should be studied individually in each case. The following estimates, in gallons per capita daily, have been made in connection with the design of sewerage works:

Cincinnati, Ohio	50
Fitchburg, Mass., maximum	81
Louisville, Ky	57
Neponset Valley intercepter, Mass	25
Milwaukee, Wis	57
Passaic Valley intercepter, N. J., maximum	38
Paterson, N. J., maximum	18
Providence, R. I., maximum	42

The variation in the hourly rate of flow from a typical industrial district with some residences, in Cincinnati, is shown by

curve C, Fig. 11. It had an area of 294 acres and a population of 19.1 persons per acre. The curve is based on 3 days' gagings during which the average rate was 6,787 gal. per acre and the maximum rate 13,485 gal. per acre daily. This maximum is equivalent to an hourly rate of 708 gal. per capita daily, 198 per cent of the average daily rate.

39. Provision for Storm Water in Intercepting and Separate Sewers.—There is a general impression that it is wise to provide in intercepting sewers for a small quantity of storm water, expressed often as being sufficient for the "first flushings" of street surfaces and sewers. This impression is based upon the assumption that there are accumulations of sewage sludge in the sewers and quantities of filth on the streets which will be immediately flushed into the intercepting sewers with the first run-off due to rain. In some sewers laid on very flat grades, or where sewers have settled or have been built with depressions in them, there may be such deposits, but where sewers are laid on grades which give satisfactory velocities such deposits are believed to be exceptional. Where deposits do occur they are generally found to consist largely of sand and other heavy detritus, dropped by the current as the velocity of flow decreased with the decrease in volume and depth of flow in the sewer. It is apparent, therefore, that these deposits will not be picked up again by the current until a substantial increase in rate of discharge is reached.

Intercepters are fed by trunk sewers serving rather large districts. Considerable time is required to flush the major part of the district systems to the intercepters, during which a large flow is likely to reach them from the nearer portions. Unless considerable surplus capacity is provided, the intercepters will often be running full before the flushings from much of the tributary area can reach them. Therefore, too much stress should not be laid on their ability to care for the "first flushings," although as ordinarily designed they can accomplish something in this direction.

Assuming that the average daily flow of sewage is 100 gal. per capita and that the capacity of the intercepter is 300 gal. per capita, there will be a surplus capacity available for "first flushings" equivalent to twice the average flow of sewage, if such flushings come at a time when the flow is at the average rate. The maximum rates of flow generally occur in the spring

when ground water is high, and at other times there will always be some surplus capacity. Furthermore, as intercepters are usually designed to be adequate for many years, there will be a considerable excess capacity during the earlier years, although this should not usually be counted upon to care for storm water, for it is a gradually diminishing allowance accompanied by a gradually increasing need, if need there should be.

Under the foregoing conditions the sewage will be diluted to one-third its normal strength. If consideration be given to the excess of water used and to the quantity of ground water which leaks into the sewers in this country, giving a more dilute sewage than that of England, it is evident that the dilution provided is about double that of the standard of the Royal Commission on Sewage Disposal (1908), which recommends treatment of about three times the dry-weather flow in the ordinary tanks; but the Commission also recommends provision of storm stand-by tanks for treatment of such additional flow as may be brought to the plant.

The foregoing outline of a rational method of estimating the quantity of sewage to be provided for applies to the design of a number of structures such as intercepters, pumping equipment, grit chambers, and large trunk sewers, and the units discussed are for maximum rates of flow when the sewers are full.

These principles are also applied in fixing the basis of design for a separate system of sewers, or a system from which it is intended that storm water shall be excluded. It is important to note, however, that in practice it is not uncommon for a considerable amount of surface run-off to find its way into manholes, and for numbers of storm-water roof conductors to be connected to separate sewers, in spite of rules or ordinances prohibiting such connection. It is wise, therefore, in separate sewer design, to make some additional allowance for storm water in the maximum rate of sewage flow.

40. Total Quantity of Sewage.—Having given consideration to the population, area, and average and maximum rates of flow to be expected in residential, mercantile, industrial, and park districts, it is next necessary to combine the different elements to arrive at an estimate of the quantity of sewage for which provision should be made for the entire community, or the portion of it which may be served by a trunk or intercepting sewer. It

TABLE 15.—POPULATIONS AND AREAS OF CINCINNATI SEWER DISTRICTS AS OF 1912 AND 1950 OHIO RIVER DISTRICT
(Part of table to illustrate form and method)

Number	Sewer district, name	As of 1912			As of 1950								
					Area in acres								
		Area inside city (acres)	Population	Density, persons per acre[1]	Total area of sewer district	Parks and cemeteries	Railroad yards	Industrial area	Mercantile area	Residential area	Total area excluding parks, cemeteries, R. R. yards	Density, persons per acre[1]	Population
50	Waldon St.	100.8	1,168	11.6	100.8	.....	.....	22.0	.....	78.8	100.8	15.0	1,510
51	Eggleston Ave.	1,509.8	50,435	39.5	1,509.8	203.0	29.2	271.8	161.0	844.8	1,277.6	49.3	63,000
52	Butler St.	12.3	400	32.5	12.3	.....	.....	12.3	.....	....	12.3	39.8	490
53	Pike St.	10.5	446	42.5	10.5	.....	.....	10.5	.....	....	10.5	40.0	420
54	Lawrence St.	15.1	368	24.4	15.1	.....	.....	15.1	.....	....	15.1	39.7	600
55	Ludlow St.	14.4	928	64.5	14.4	.....	.....	14.4	.....	....	14.4	65.3	940
56	Broadway	26.0	1,959	75.3	26.0	.....	.....	12.0	14.0	....	26.0	80.0	2,080
57	Sycamore St.	37.9	1,702	44.9	37.9	.....	.....	8.0	29.9	....	37.9	60.0	2,270
58	Main St.	28.4	540	19.0	28.4	.....	.....	5.0	23.4	....	28.4	25.0	710
59	Walnut St.	36.9	421	11.4	36.9	.....	.....	6.0	30.9	....	36.9	10.0	370
60	Vine St.	[illegible]3.7	308	9.1	33.7	.....	.....	7.0	23.7	....	33.7	10.1	340
	Totals of districts 40 to 80 inclusive	[illegible]4.3	238,794	16.0	17,266.5	540.2	118.0	2,092.0	721.2	13,795.1	16,608.3	18.3	303,826

[1] In computing density of population, areas of parks, cemeteries, and railroad yards are deducted from total area of sewer district.

will simplify the explanation of the method of estimating and serve to summarize the whole discussion, if an illustration from actual practice is given. For this purpose the studies made in Cincinnati, Ohio, in 1913, already alluded to, may be taken. Three main intercepter districts were planned and designated as the Duck Creek, Ohio River, and Mill Creek districts from the names of the water courses along which the intercepters were to be constructed.

Having first studied the local conditions and estimated the probable growth of the city as a whole, both in population and area, during the assumed "economic period of design," consideration was given to the distribution of population and area among the several sewer districts. The respective areas, as dictated by topography, were indicated upon maps and measured with the planimeter. A large map was then prepared upon which were indicated the outlines of the residential, mercantile and industrial areas and parks, railroad yards and cemeteries. The portions of each coming within each sewer district were measured and tabulated as in Table 15, together with the estimates of future population.

Consideration was next given to the quantity of sewage for which provision should be made, the units of maximum rate of flow in intercepters adopted after a study of local conditions and of all data available being given in Table 16.

Table 16.—Unit Quantities of Flow in Intercepters Assumed for Cincinnati, Ohio

Residential areas	
Sewage	135 gal. per capita per day
Ground water	750 gal. per acre per day
Mercantile areas	
Sewage (resident population)	135 gal. per capita per day
Additional allowance for character of development	40,000 gal. per acre per day
Ground water	750 gal. per acre per day
Industrial areas	
Sewage (resident population)	135 gal. per capita per day
Industrial wastes	9,000 gal. per acre per day
Ground water	750 gal. per acre per day
Parks, railroad yards and cemeteries	
Ground water	750 gal. per acre per day

The results obtained by the computations are illustrated by Table 17 and are summarized for the whole city in Table 18.

TABLE 17.—ESTIMATED QUANTITY OF CINCINNATI SEWAGE TO BE PROVIDED FOR AT MAXIMUM RATE OF FLOW AS OF 1950
OHIO RIVER DISTRICT
(Part of table to illustrate form and method)

Number	Sewer district, name	Estimated future population	Area in acres: Total area	Area in acres: Industrial area	Area in acres: Mercantile area	Million gallons per day: Water supply reaching sewers 135 g.c.d.	Million gallons per day: Ground water 750 g.a.d.	Million gallons per day: Industrial sewage 9,000 g.a.d.	Million gallons per day: Mercantile sewage 40,000 g.a.d.	Estimated total quantity: m.g.d.	Estimated total quantity: c.f.s.	Cumulative quantities: m.g.d.	Cumulative quantities: c.f.s.
50	Waldon St.	1,510	100.8	22.0	.....	0.204	0.076	0.198	......	0.478	0.7	14.061	21.7
51	Eggleston Ave.	63,000	1,509.8	271.8	161.0	8.505	1.132	2.446	6.440	18.523	28.7	32.584	50.4
52	Butler St.	490	12.3	12.3	.....	0.066	0.009	0.111	.....	0.186	0.3	32.770	50.7
53	Pike St.	420	10.5	10.5	.....	0.057	0.008	0.094	.....	0.159	0.2	32.929	51.0
54	Lawrence St.	600	15.1	15.1	.....	0.081	0.011	0.136	.....	0.228	0.3	33.157	51.3
55	Ludlow St.	940	14.4	14.4	.....	0.127	0.011	0.130	.....	0.268	0.4	33.425	51.7
56	Broadway	2,080	26.0	12.0	14.0	0.281	0.020	0.108	0.560	0.969	1.5	34.394	53.2
57	Sycamore St.	2,270	37.9	8.0	29.9	0.306	0.028	0.072	1.196	1.602	2.5	35.996	55.7
58	Main St.	710	28.4	5.0	23.4	0.096	0.021	0.045	0.936	1.098	1.7	37.094	57.4
59	Walnut St.	370	36.9	6.0	30.9	0.050	0.028	0.054	1.236	1.368	2.1	38.462	59.5
60	Vine St.	340	33.7	7.0	26.7	0.046	0.025	0.063	1.068	1.202	1.9	39.664	61.4
	Totals of districts 40 to 80 inclusive	303,826	17,266.5	2,092.0	721.2	41.017	12.950	18.828	28.848	101.643	157.2		

In this table, g.c.d. is an abbreviation of gallons per capita daily: g.a.d. is an abbreviation of gallons per acre daily: m.g.d. stands for million gallons daily, and c.f.s. for cubic feet per second.

TABLE 18.—ESTIMATED TOTAL QUANTITY OF SEWAGE TO BE PROVIDED FOR AT MAXIMUM RATE OF FLOW IN THREE INTERCEPTER DISTRICTS, CINCINNATI, OHIO, AS OF 1950

Intercepter district	Estimated future population	Area in acres: Total area	Area in acres: Industrial area	Area in acres: Commercial area	Million gallons per day: Water supply reaching sewers 135 g.c.d.	Million gallons per day: Ground water 750 g.a.d.	Million gallons per day: Industrial sewage 9,000 g.a.d.	Million gallons per day: Commercial sewage 40,000 g.a.d.	Estimated total quantity: m.g.d.	Estimated total quantity: c.f.s.	Quantity in units of: Gallons per acre per day	Quantity in units of: Gallons per capita per day
Mill Creek	308,664	52,740.1	4,863.3	.....	41.670	39.555	43.770	......	124.995	193.4	2,370	405
Ohio River	303,826	17,266.5	2,092.0	721.2	41.017	12.950	18.828	28.848	101.643	157.2	5,885	334
Duck Creek	99,320	12,690.9	1,228.5	.....	13.408	9.518	11.056	......	33.982	52.6	2,680	342
Total	711,810	82,697.5	8,183.8	721.2	96.095	62.023	73.654	28.848	260.620	403.2	3,150	366

The rates in all cases are the highest anticipated at times when it will be necessary to intercept the sewage or ultimately to treat it. They are influenced by the smoothing-out effect of the differences in time of entrance of sewage from lateral sewers into trunk sewers and from trunk sewers into intercepters. The ground-water allowance is low, because of the topography, which assures the location of most of the sewers above the elevation of ground water.

The allowances for flow in the several districts and for the entire city are given in Table 19. There are substantial differences in the allowances for the several intercepters, more than twice as large a flow per acre being provided for in the Ohio River intercepter as in either of the others.

TABLE 19.—ESTIMATED UNIT QUANTITIES OF SEWAGE TO BE PROVIDED FOR AT A MAXIMUM RATE OF FLOW IN THREE MAIN INTERCEPTER DISTRICTS, AS OF 1950, CINCINNATI, OHIO

	Duck Creek intercepter			Ohio River intercepter			Mill Creek intercepter			Whole city		
	Gal. per acre per day[1]	Gal. per capita per day	Per cent	Gal. per acre per day[1]	Gal. per capita per day	Per cent	Gal. per acre per day[1]	Gal. per capita per day	Per cent	Gal. per acre per day[1]	Gal. per capita per day	Per cent
Sewage from resident population	1,057	135.0	39.5	2,375	135.0	40.4	790	135	33.4	1,162	135.0	36.9
Additional allowance, mercantile areas	.....	.....	.....	1,670	95.0	28.4	.....	.....	.....	349	40.6	11.1
Additional allowance, industrial areas	873	111.2	32.5	1,090	62.0	18.5	830	142	35.0	890	103.3	28.2
Ground water	750	95.8	28.0	750	42.0	12.7	750	128	31.6	750	87.1	23.8
Totals	2,680	342.0	100.0	5,885	334.0	100.0	2,370	405	100.0	3,150	366.0	100.0

[1] Total area of district.

Industrial sewage, based upon 9,000 gal. per acre of industrial area; mercantile sewage, upon 40,000 gal. per acre of mercantile area; ground water, upon 750 gal. per acre of total area; domestic sewage, upon 135 gal. per capita.

Problems

1. The population of Reading, Pa., in 1920 was 107,784. Assuming a uniform rate of growth of 12 per cent each 10 years, what will be the population in 1960?

2. Draw a reasonably smooth curve through the U. S. Census population figures from 1800 to 1920 for Worcester, Mass., given herewith and extend the curve to 1960. What will be the probable population in 1960?

1800	2,411	1870	41,105
1810	2,577	1880	58,291
1820	2,962	1890	84,655
1830	4,173	1900	118,421
1840	7,497	1910	145,986
1850	17,049	1920	179,754
1860	24,960		

3. The following are census figures for the cities indicated taken at 10-year intervals:

	1870	1880	1890	1900	1910	1920
Providence, R. I.	23,171	41,513	60,666	68,904	104,857	132,146
Worcester, Mass.	24,960	41,105	58,291	84,655	118,421	145,986
Richmond, Va.	27,570	37,910	51,038	63,600	81,388	85,050
Des Moines, Iowa	12,035	22,408	50,093	62,139	86,368	126,468
Albany, N. Y.	24,209	33,721	50,763	62,367	69,422	90,758
Topeka, Kans.	......	......	31,007	33,608	43,684	50,022

Without regard to the propriety of using these data, compare the rate of growth of Topeka graphically with those of the above cities, and estimate its probable population in 1945 on the basis of such comparison.

4. From the data in Problem 3, compute and plot the relation between size and percentage increase per decade for the five cities after their populations reached the 1920 population of Topeka, and with these data estimate the probable population of Topeka in 1945.

5. Assuming that the ten year arithmetical rate of increase of population of Topeka, Kans., will be the same in the future as for the 1910 to 1920 decade, what will be its population in 1945?

6. Using the average curve in Fig. 6 and with the 1920 population of Worcester as given in Problem 2, compute the population in 1960.

7. If the Milwaukee sewage flows into aeration chambers having capacity for 6 hours' flow at the average rate, what would be the average detention period (time of flow through the chambers): from 7 A.M. to 12 noon? from 8 P.M. to 6 A.M.? Use the dotted line in Fig. 8 for hourly variations in flow for the Milwaukee sewage.

8. Assuming the density of population as shown in Fig. 9 for Lynn, Mass., find the maximum rate of discharge of sewage to be expected in 1950 from an area of 2,400 acres if the average water consumption is 115 gal. per capita per day and the expected flow of sewage is 90 per cent of the water consumption and the hourly variations in flow in accordance with the smooth curve in Fig. 8. Assume the maximum rate of ground-water leakage at 4,000 gal. per acre per day.

9. An intercepter is planned to receive the discharge of three sewers each having flow characteristics as in Curve *A*, Fig. 11. Sewer *A* serves 5,000 people, Sewer *B*, 3,500 and Sewer *C*, 2,400. Velocities in the intercepter

will be such that the flow from C will reach the point of interception with B in two hours and the combined flow will require another two hours to reach the junction with A. Determine the required capacity of the intercepter below its junction with A.

10. If a reconsideration of the design of the Cincinnati intercepting sewers for which data are given in Tables 15 and 16, indicated that an average 1950 density of 20.0 persons per acre might be used for all areas and that 100 gal. per capita per day will reach the sewers as domestic sewage from the water supply, with ground water and other data as in the tables, compute in tabular form the maximum rate of sewage flow to be expected from districts 50 to 54, inclusive, in m.g.d. and c.f.s.

CHAPTER III

QUANTITY OF STORM WATER

The rate at which storm water, including melted snow and ice, will flow from the surface of the ground into drains and combined sewers must be estimated before designing these conduits. The quantity of storm water is usually so much larger than the quantity of sewage that the sizes of combined sewers are determined generally by the rates of rainfall and corresponding run-off. The amount and intensity of rainfall can be measured. The run-off or part of the rainfall which reaches the drains or combined sewers depends upon the ratio of the impervious to the relatively absorbent surface, the intensity and duration of the rainfall, the character, shape, and slope of the drainage area, and other considerations. It is manifest that a smaller proportion of a light rainfall on a dry sandy tract will reach the sewers than of a heavy rainfall on a clay tract of the same size and topography.

41. Rain Gages.—The type of rain gage that has been most widely used consists of a cylinder as shown in Fig. 12, 8 in.

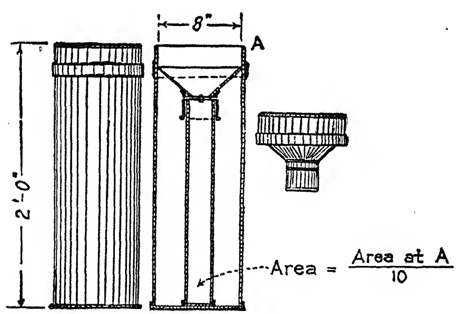


Fig. 12.—Standard rain gage—U. S. Weather Bureau.

in diameter and about 2 ft. high. It is capped with an 8-in. sharp crested funnel emptying into an inner cylinder having a cross-section of one-tenth that of the funnel. The depth of rain collected in this cylinder is measured each day with

a graduated cedar stick. The depth of precipitation is thus magnified ten times and readings to one-hundredth of an inch are taken. Should the capacity of the inner cylinder be exceeded, the overflow is retained in the outer cylinder and can be measured.

In sewerage engineering the rate of rainfall during periods just long enough to produce maximum run-off conditions is more important than the total daily rainfall, for reasons to be discussed later. Such rates can only be measured practically by automatic recording rain gages, and on account of the small number of these instruments in use until recently, there are few long-time records of this character.

42. Recording Rain Gages.—The gages which give the records most helpful to sewerage engineers make graphical records from which it is possible to determine accurately the precipitation in time intervals as short as 2 to 5 min.

In all such instruments the rainfall entering a circular collecting area or opening, from 8 to 14.85 in. in diameter in different instruments, passes to an operating mechanism, of which there are three general types. The first type receives the water in a container supported on a weighing device which actuates recording mechanism that moves a pen pressing on a moving chart. In the second type, the rain water is delivered to tilting pans, which tip when full and thus discharge their contents. Every act of tipping actuates an electrical or mechanical device which makes a record on a chart moved by clockwork, and as the quantity of water discharged by each tipping is known, the rate of rainfall is thus determined. In the third type, the water flows into a container in which there is a float with a vertical arm carrying a pen. This pen records the height of water in the container on a chart moved by clockwork, and the container is usually of such size that the float will rise at a faster rate than the rate of rainfall.

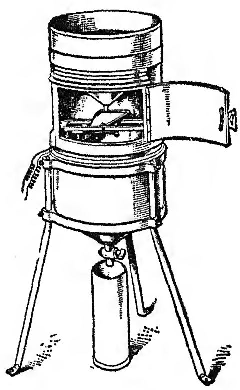


Fig. 13.—Friez tipping bucket gage.

Fig. 13 illustrates a tipping-type of gage made by Julien P. Friez and Son, Baltimore. The rain is collected in a funnel 12 in. in diameter and conducted through a tube into a bucket

with two compartments, each holding the equivalent of 0.01 in. of rain. The bucket is supported on trunnions in such a manner that as soon as a compartment is full the bucket tips, the compartment is emptied and the other compartment brought into position for filling. Each time the bucket tips, it closes an electric circuit which causes a pen to record a step on a chart carried by a revolving cylinder, Fig. 14. The record does not represent directly the progress of the storm, the motion of the pen being reciprocating, up for 0.05 in. and down 0.05 in.

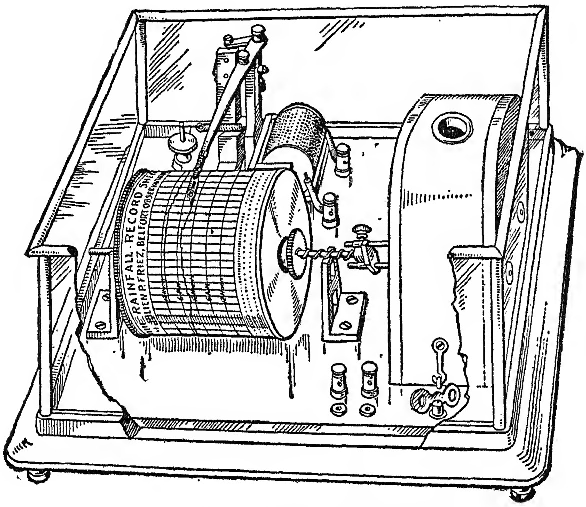


FIG. 14.—Register for Friez gage.

The time-scale of this chart is 2½ in. to an hour. The amount of rainfall is indicated, not by measurement on the chart, but by counting the number of steps, or of "flights" of 5 steps each. It is therefore possible to determine the rates of rainfall from this record with a good degree of precision. The Weather Bureau carefully investigated the accuracy of the instrument and determined that on account of the appreciable time required for the bucket to tip, the error due to the inflow of water into a compartment already full before the bucket could tip and present the empty compartment is sufficient to produce an error of about 5 per cent at times of very heavy rain.

One of the oldest and most satisfactory gages, devised by Desmond FitzGerald, is shown in Fig. 15, and the chart made

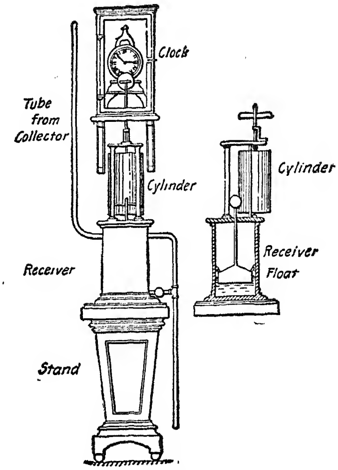
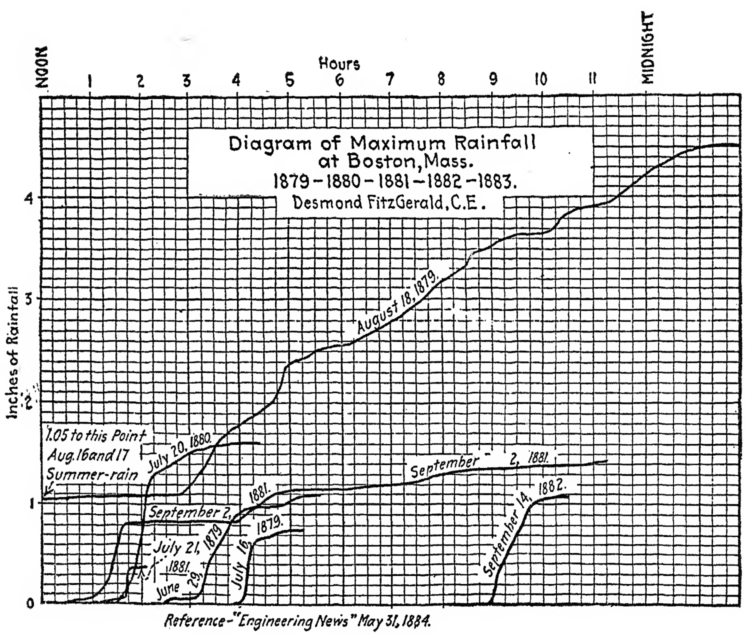


FIG. 15.—FitzGerald gage.



The complete diagram is 24 in. long.

FIG. 16.—Chart from FitzGerald gage.

from it is ruled like the diagram in Fig. 16. The charts move from right to left, however, and not as indicated. The rain is collected in a funnel 14.85 in. in diameter, and conducted through a tube into a receiver containing a float. The diameter of the receiver is such that 1 in. of rain causes the float to rise 2 in. The float carries a pencil bearing directly upon the chart carried by a revolving cylinder. This cylinder is of such a size that a chart 24 in. long is revolved once every day so that the time-scale is 1 in. per hour. It is therefore possible to determine rates of precipitation with fair accuracy.

Gages should be set[1] at least 50 ft. from objects which may cause wind currents interfering with accurate measurement, and the collector ring or opening should be within 30 in. of the ground. It is often difficult to comply with the latter condition, and if the collecting ring is at a considerable elevation above ground, it may receive a markedly smaller quantity of water than if placed within the standard distance of the ground. If it is impossible to place the collecting ring practically at ground level, a standard non-recording rain gage should be maintained near the automatic gage and the records of the latter adjusted to agree with those of the standard gage.

43. Intensity of Precipitation.—In a general way the intensity of precipitation varies inversely with the duration of the downpour, or in other words, very heavy showers do not last as long as rains of lesser intensity. Intensity is expressed in terms of inches per hour; that is, if 1 in. of rain falls in 20 min., the rate or intensity of precipitation is 3 in. per hour.

By careful measurements of a gage chart the rates of precipitation for such periods as 5, 10 and 15 min. during the storm can be ascertained. When such records are available for a considerable period of years they furnish a fair indication of the intensities which may be expected on the average during a term of years approximately equal to the period of record. They may not, however, cover the extreme conditions likely to occur, or they may include a great storm due to occur only once on the average in a much longer period.

44. Development of Time-intensity-frequency Curves.—If automatic records of intense rainfalls are available for a sufficient period, say 20 years, so that they may be assumed to include

[1] Circular E, Instrument Division, U. S. Weather Bureau, on Measurement of Precipitation, gives valuable information on this important subject.

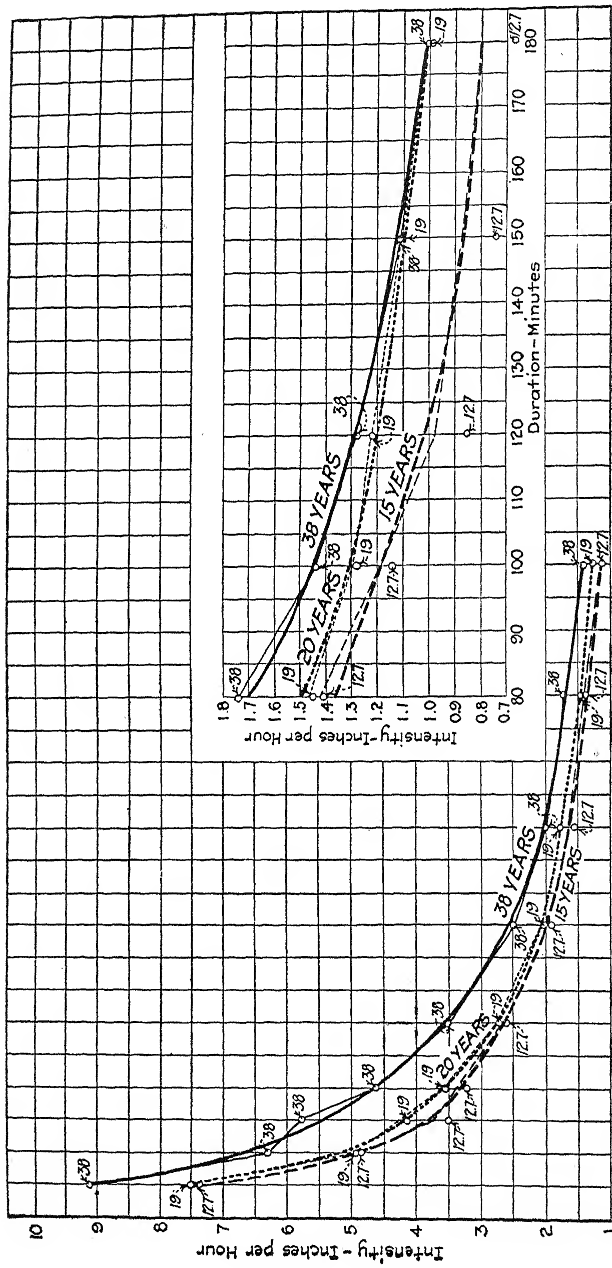


FIG. 17.—Rainfall curves for Chestnut Hill Reservoir (Boston), from 38-year record (1879–1916).

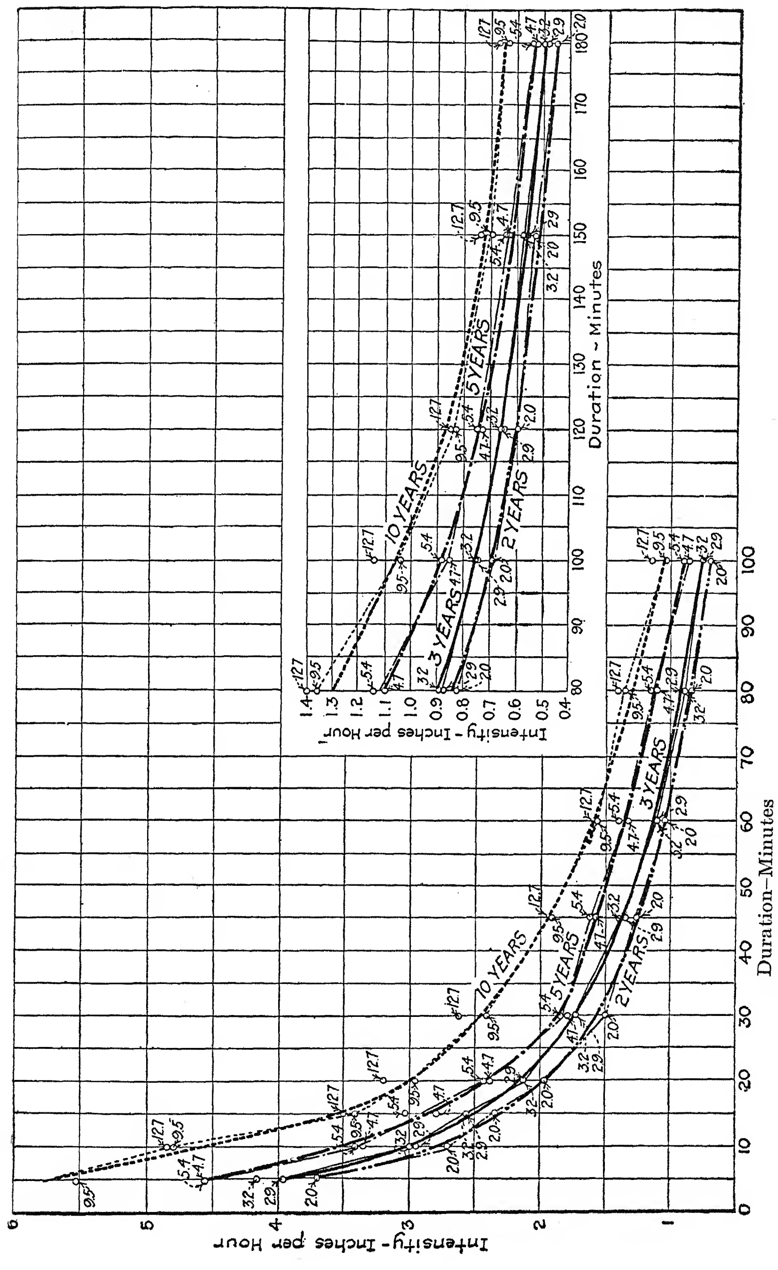


FIG. 18.—Rainfall curves for Chestnut Hill Reservoir (Boston), from 38-year record (1879–1916).

all the weather conditions which are ordinarily experienced, then the maximum observed intensity for any duration of rain may be taken as that likely to be equalled or exceeded once in that period, in this case once in 20 years; the second has been equalled or exceeded twice in the period of record, or on the average once in 10 years, and may be assumed to have a frequency of 10 years. Similarly, intensities likely to occur in any time interval shorter than the period of record may be obtained.

If then the highest 10 points for each time interval (duration of downpour) be plotted, the lines connecting the several points of the same rank will represent the intensities which have been *equalled or exceeded* on the average once in 20 years, 10 years, 6⅔ years, 5 years, and other intervals, say, down to once in 2 years. For convenience, the corresponding rainfall rates are said to have frequencies of 20 or 10 years, and the like. Judging the future by the past, and smoothing out the curves so as to eliminate obvious inconsistences (as for instance when a point is so far in excess of others as to indicate an intensity unlikely to recur in a much longer period), the resulting curves may reasonably be taken as time-intensity curves of various degrees of frequency.

Examples of such curves are shown in Figs. 17, 18 and 19.

Obviously the lower curves are less likely to be in error than the outermost or enveloping curve; and the longer the period of record, the better the series of curves. Some of the maximum points in a record, say of 30 years, for example, may diverge so much from the others as to indicate that they really belong on a curve of much less frequency, such as a 50- or 100-year curve. On the other hand, the highest two or three curves may be so close together as to indicate that the highest is probably below the true average curve for the stated period of years.

In an analysis of this kind there is no reliable way of estimating the probable frequency of such abnormal data. It is customary in such cases to omit from consideration any points which are so far above the others as to indicate that they really represent the maximum for a much longer period of time than that covered by the record.

Plotting the data on logarithmic co-ordinate paper for each duration may be employed to assist in forming a judgment of frequency of storms not in conformity with the bulk of the data.

If the curves for the various durations are in reasonable agreement with the plotted points, it may be possible to extend them and to construct a curve of probable relations corresponding to a frequency much less than the term of the observations; for instance, a 50-year curve might be based upon records covering 20 years. The dangers of relying too much upon such extrapolated curves are obvious.

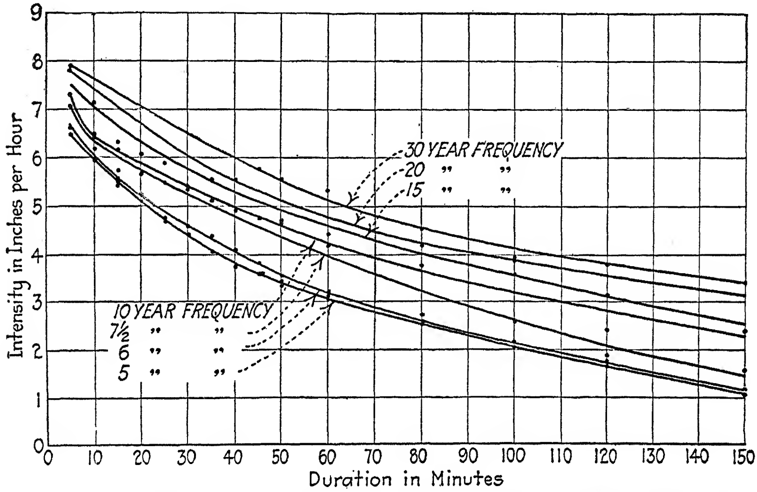


FIG. 19.—Rainfall curves for Galveston, Tex., from 30-year record (1890–1919).

The selection of the rainfall curve to be adopted as the basis for design in any community is an important matter. While it might be desirable to provide drainage channels adequate for the greatest storms of which there are local records, such a course would usually require large expenditures which in many cases would be prohibitive, particularly if proper provision be made for increase in the proportion of impervious area likely to take place within the next 30 to 50 years in a rapidly growing community. It may be necessary, therefore, to reduce the allowances, and one of the most logical ways in which this can be done is by the adoption of a rainfall curve likely to be equalled more frequently, as, for example, once in 15, 10 or even as low as 5 years. In making the selection, consideration should be given to the several economic conditions of the case, especially the funds reasonably available for construction and the probable

amount of damage and inconvenience caused by occasional flooding.

45. Form of Rainfall Curve.[1]—Many equations of rainfall curves have been written in the form $i = a/(t + b)$. A curve of this form often expresses the actual observations with a fair degree of accuracy for rains of 10 min. to 2 hr. duration. Beyond these time limits the results are generally too low. In some cases the exponential equation, $i = a/t^b$ has been found satisfactory. The exponent b is usually between 0.5 and 0.7. The form $i = a/(t + b)^c$ sometimes agrees with the data better than any other.

It is possible that a single form of equation may be applicable to a large area, as for instance to the northeastern section of the United States; that variation of the constants b and (or) c will adjust it for a particular locality, and of a for frequency of occurrence. It will be necessary to analyze a considerable number of long-time records obtained in various parts of the country before this can be determined.

46. Intensity of Rainfall at Louisville, Kentucky.—The United States Weather Bureau has maintained a rain gage in Louisville since 1872. Since 1894 a registering automatic rain gage has been in use. The records from 1894 to 1897 inclusive for excessive rainfalls[2] are recorded in the Monthly Weather Review, the maximum amount being given for the 5-, 10-, and 60-min. intervals. Since 1897, records of accumulated amounts have been published for several time intervals, after the beginning of intense rates. In this study, the 24 years' record for the period 1897–1920 is used.

The maximum amounts and intensities are given in Table 20, arranged in the order of their magnitude and number of occurrences for the periods indicated, regardless of the storm in which they occurred; that is, the amounts and intensities shown for a given number of occurrences did not necessarily occur during a single storm. For example, the highest amount for 5-min. duration occurred during the storm of July 15, 1919; for 10-, 15-, 20-, 25-, 30-, 45-, and 60-min. during the storm of Aug. 29, 1917; and for durations longer than 60 min. during the storm of Aug. 8, 1898. Therefore a sewer system, if designed to care

[1] In this discussion, i represents intensity of rainfall in in. per hour, and t, duration in minutes; a, b and c are constants.

[2] *Excessive rainfalls*, as defined by the U. S. Weather Bureau, include all rates of precipitation in excess of $(0.01t + 0.20)$, *i.e.*, time in minutes (taken as hundredths of an inch) plus 0.20. Thus 0.25 in. in 5 min., 0.80 in. in 60 min., and 1.40 in. in 120 min., are excessive rainfalls.

TABLE 20.—INTENSITY, DURATION, AND FREQUENCY OF RAINFALL, AT LOUISVILLE, KY.

(Twenty-four years, 1897–1901)

Number of times equalled or exceeded		1	2	3	4	5	6	7	8	9	10
Average frequency, years		24	12	8	6	4 4/5	4	3 3/7	3	2 2/3	2 4/10
Duration, minutes											
5	Amount[1]...	0.77	0.74	0.61	0.57	0.49	0.48	0.45	0.42	0.41	0.38
	Intensity[2]...	9.24	8.88	7.32	6.84	5.88	5.76	5.40	5.04	4.92	4.56
10	Amount....	1.15	1.14	0.87	0.76	0.69	0.67	0.66	0.65	0.64	0.61
	Intensity...	6.90	6.84	5.22	4.56	4.14	4.02	3.96	3.90	3.84	3.66
15	Amount....	1.40	1.33	1.05	1.00	0.95	0.90	0.84	0.78	0.77	0.77
	Intensity...	5.60	5.32	4.20	4.00	3.80	3.60	3.36	3.12	3.08	3.08
20	Amount....	1.59	1.44	1.24	1.16	1.14	1.09	0.96	0.96	0.93	0.90
	Intensity...	4.77	4.32	3.72	3.48	3.42	3.27	2.88	2.88	2.79	2.70
25	Amount....	1.68	1.49	1.45	1.33	1.32	1.14	1.10	1.05	1.03	1.00
	Intensity...	4.03	3.58	3.48	3.19	3.17	2.74	2.64	2.52	2.47	2.40
30	Amount....	1.72	1.71	1.54	1.44	1.44	1.24	1.13	1.10	1.09	1.09
	Intensity...	3.44	3.42	3.08	2.88	2.88	2.48	2.26	2.20	2.18	2.18
45	Amount....	1.82	1.81	1.69	....	1.47	1.45	1.39	1.29	1.27	1.24
	Intensity...	2.43	2.41	2.26	....	1.96	1.93	1.85	1.72	1.69	1.65
60	Amount....	2.00	1.93	....	1.73	1.53	1.47	1.41	1.39	1.36	1.33
	Intensity...	2.00	1.93	....	1.73	1.53	1.47	1.41	1.39	1.36	1.33
80	Amount....	2.43	2.38	1.99	....	1.58	1.57	1.57	1.57	1.53	1.49
	Intensity...	1.82	1.79	1.49	....	1.19	1.18	1.18	1.18	1.15	1.12
100	Amount....	2.70	2.44	2.02	....	1.67	1.65	1.61	1.61	1.60	1.58
	Intensity...	1.62	1.46	1.21	....	1.00	0.99	0.97	0.97	0.96	0.95
120	Amount....	2.72	2.46	2.05	....	1.73	1.72	1.72	1.66	....	1.62
	Intensity...	1.36	1.23	1.03	....	0.87	0.86	0.86	0.83	....	0.81
180	Amount....	3.01	2.49	2.28	2.13	2.09	1.86	1.85	1.85	1.80	1.74
	Intensity...	1.00	0.83	0.76	0.71	0.70	0.62	0.62	0.62	0.60	0.58
240	Amount....	3.16	....	2.48	2.25	2.14	2.10	2.03	1.93	1.90	1.89
	Intensity...	0.79	....	0.62	0.56	0.54	0.52	0.51	0.48	0.48	0.47
300	Amount....	3.21	2.60	2.52	2.28	2.20	2.19	....	1.98		
	Intensity...	0.64	0.52	0.50	0.46	0.44	0.44	....	0.40		
360	Amount....	....	2.82	2.69	2.38						
	Intensity...	....	0.47	0.45	0.40						

[1] Amount in inches.
[2] Intensity in inches per hour.

for the runoff from rainfall intensities as shown for a given number of occurrences, would be likely to be surcharged only in part by a single storm in which intensities corresponding to those of a smaller number of occurrences were experienced.

The first line in the table shows the order or number of occurrences, and applies to the amounts and intensities given in the columns below the numbers. The figures represent the number of times that the intensities and amounts for the different periods have been equalled or exceeded in the 24 years covered by the record.

The second line shows the frequency in years and is equal to the length of the record divided by the number of occurrences.

In column 1 is the highest amount in inches, and the corresponding intensity in inches per hour for the duration stated. In column 2 is the second highest amount in inches and the corresponding intensity in inches per hour for the duration stated, and so on for the other columns.

It will be noted that no data are given for the storm of 360-min. duration and occurring once in the period of the record, although the storm occurring twice is shown. Similarly, for 45-min. duration, storms occurring three and five times are shown, but none for four occurrences. In these points, the table departs from the data obtained from the records, because the 360-min. precipitation tabulated as of two occurrences was actually experienced but once, and the other blanks indicate similar shifts of the record data. But if the cumulative amounts of rain be considered, it is obvious that the typical storm of 24-year frequency could not have yielded 3.21 in. in 300 min. and a smaller amount, 2.82 in., in 360 min. The quantity for 360 min. must be at least 3.21 in., unless the amount for 300 min. is abnormal and should be reduced, and the other figures give no indication that this is the case. The amount of 2.82 in., is, therefore, shifted to the column of amounts equalled or exceeded twice, subject to the same criterion when examining the data in that column. In a similar way, other figures have been shifted into columns later than their actual places, before utilizing the figures for drawing curves.

It sometimes happens that when two nearly contiguous downpours with moderate intensity of precipitation between them are treated as a single storm, somewhat higher intensities will be found for a given duration than for a shorter duration.

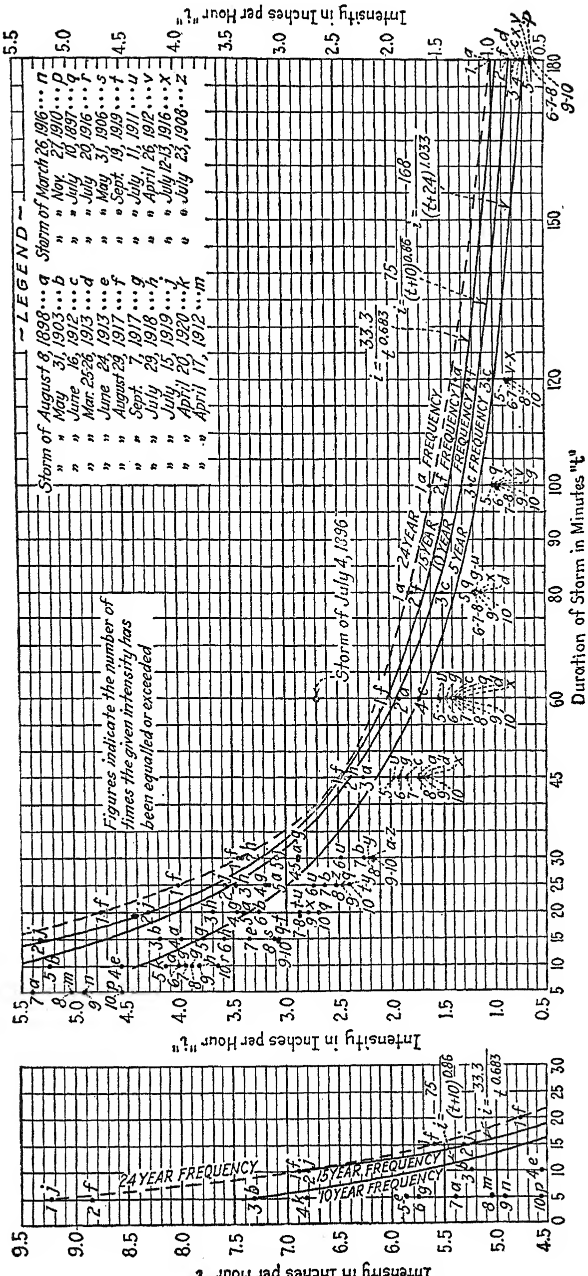


FIG. 20.—Intensity, duration and frequency of rainfall, Louisville, Ky., from U. S. Weather Bureau records 1897–1920 (24 years) August, 1921.

This is very exceptional, however, and is not in accordance with the general relation which exists between intensity and duration of rainfall.

From the data in Table 20, Fig. 20 has been plotted with intensities in inches per hour as ordinates, and durations in minutes as abscissas. On this figure is also shown a point plotted for 2.70 in. per hour and for a duration of 60 min., and marked "Storm of July 4, 1896." As can be seen, this storm lies far above that of any in the records of 1897 to 1920, inclusive. It may, therefore, be inferred that this storm is of very unusual character and will recur only at very long intervals. It is quite certain that the city would not be justified in building all its sewers of a capacity sufficient to carry off the rainfall from a storm of such magnitude. The data for this storm are not as complete as for the other storms, as the maximum intensities for the 5-, 10-, and 60-min. periods only are given. Table 21 gives the data on this and other excessive rains reported in the *Monthly Weather Review* for 1895 and 1896.

TABLE 21.—AMOUNTS AND RATES OF EXCESSIVE RAINFALL AT LOUISVILLE (INCHES) 1895–1896

Date	Duration					
	5 min.		10 min.		60 min.	
	Amount	Intensity	Amount	Intensity	Amount	Intensity
June 4, 1895.....	0.50	6.00	0.55	3.30		
June 20, 1895....	....	....	....	....	0.87	0.87
June 23, 1896....	0.55	6.60	1.00	6.00	1.37[1]	1.37[2]
July 4, 1896.....	0.55	6.60	1.05	6.30	2.70	2.70
July 20–21, 1896.	....	....	....	....	1.29	1.29

[1] Duration 36 min.
[2] Rate for 36 min. = 3.80.

It is recognized that were the data for these storms complete, they might alter the arrangement in Table 20 and change the curves slightly. Because of the paucity of data, however, these storms have not been included in the analysis of the rainfall.

On Fig. 20 is shown the 24-year frequency curve, based upon the actual records. Curves for frequencies of 15, 10, and 5 years have also been deduced from the data and are shown on this figure.

The equation $i = 33.3/t^{0.683}$ gives results which correspond closely with intensities which may be expected to be equalled or exceeded on the average once in a period of 15 years. The 10-year curve is fairly well represented by the equation $i = 75/(t + 10)^{0.86}$. The 5-year curve is represented by the equation $i = 168/(t + 24)^{1.033}$.

It appears that there is very little difference between the 15- and the 10-year curves, especially for periods of 20 to 60 min., which are frequently used in the design of sewers and are, therefore, particularly significant. The variation is less than 10 per cent, so there would probably be no difference in the sizes adopted for sewers, whichever curve were used. It is of interest, however, to notice that within the significant period from 20 to 60 min., the 10-year curve is from 15 to 20 per cent higher than the 5-year curve. The record indicates that for periods of time greater than 30 min., sewers designed by the 5-year curve would be surcharged, on the average, three times, and if designed by the 10-year curve, twice in a period of 24 years.

The 15-year frequency curve is controlled by 5 storms. For durations of 5 to 25 min., inclusive, the storms of July 15, 1919, and Aug. 29, 1917, fix the location of the curve within narrow limits. For 30- and 45-min. duration the storms of July 29, 1918, and Aug. 29, 1917, determine the position of the curve. For durations from 60 min. to 180 min., inclusive, the storms of Aug. 8, 1898, and Aug. 29, 1917, determine the position of the curve. In general, the 15-year curve corresponds to the intensities experienced during the storm of Aug. 29, 1917. It lies somewhat below this storm for periods from 10 min. to 30 min., almost coincides with it for periods of 45 and 60 min., and is again a little below for durations of 80 min. and longer.

It will be noticed from Fig. 20 that the highest two intensities for a duration of 45 min. are almost coincident, and the 10-, 15-, and 24-year frequency curves are very close together. It seems probable that the city will experience an intensity for 45 min. somewhat higher than any shown by the records. This justifies the position of the 15-year curve which lies slightly above the maximum intensity which has been recorded as indicated on the figure for this duration.

47. Phenomenal Rainstorms.—Storms of extreme intensity, commonly called "Cloud-bursts," are occasionally experienced. They are usually of such rare occurrence as to be classed as

"Acts of God," for which it would not be reasonable to provide in designing storm drains. They are not to be confused with storms resulting from special topographical conditions, more particularly in mountainous country, which may produce rainfalls abnormally high according to usual standards but not unusual for the specific locality under consideration.

During 1913 New York City experienced four storms, in all of which the intensity of precipitation, practically throughout the storm, was greater than that given by the equation $i = 15/t^{0.5}$. The significant facts relative to these storms and the intensities obtained by this formula are given in Table 22.

TABLE 22.—PHENOMENAL RAINFALL IN NEW YORK CITY, 1913

Date	July 10	July 28	Sept. 5	Oct. 1	$i = \frac{15}{\sqrt{t}}$
Place	100 Broadway	Central Park	Central Park	Richmond	
t minutes	Intensity i inches per hour				
1	....	....	....	8.40	15.00
2	....	....	....	8.10	10.60
4	....	....	....	6.45	7.50
5	9.88	6.12	7.20	....	6.72
7	....	....	....	6.24	5.68
10	7.56	5.76	6.90	5.64	4.75
15	6.52	4.80	6.36	5.16	3.88
19	....	....	....	5.05	3.45
30	4.18	2.96	5.24	....	2.74
37	....	....	....	4.84	2.47
49	....	....	....	4.75	2.15
59	....	....	....	4.44	1.95
60	2.30	2.73	3.31	....	1.94
85	....	....	....	3.80	1.63
106	....	....	....	3.37	1.46
120	1.28	1.56	1.85	....	1.37
123	....	....	....	3.06	1.35

48. Distribution of Intense Rainfall.—The preceding discussion of intense precipitation and of time-intensity relations has been based upon the records of individual gages; that is, upon the rainfall at a definite point in each case considered. Where two or more gages have been placed a mile or more apart, it has been observed that, with intense storms, there is considerable difference in the rate and amount of rainfall recorded by adjacent gages. The area on which rain falls at a high rate for periods of an hour or less is comparatively small. In the drainage of large areas, there is possibility of considerable error in assuming

the intensity of rainfall to be uniform and equal to that indicated by the time intensity curve for the locality under consideration. Very little detailed information upon the distribution of intense rainfall is to be had. Few localities are even yet provided with an adequate number of recording rain gages so situated as to show the travel of and area covered by storms. The eight recording gages in New York City are distributed over an area 29 miles long and 1½ miles wide; the three in Washington are in a line approximately parallel to the Potomac River; the seven in St. Louis are in an area about 6¾ miles long by 1¾ miles wide.

In the Boston Metropolitan District there are 13 recording rain gages within 10 miles of the State House, so situated as to cover fairly well an area of about 12 by 18 miles. Records of these gages during the years 1918 to 1922 have been compared and studied by Frank A. Marston,[1] who also made a similar comparison of the records of one storm (in 1920) at New Orleans, and of a phenomenal storm at Cambridge, Ohio, in 1914.

The data given indicate that, in a general way, the average intensity of precipitation, within the limits usually met in design, over an area of 1,000 acres is about 91 per cent of the maximum; and over an area of 5,000 acres the average is about 83 per cent of the maximum. Approximate ratios of average intensity to maximum intensity for several periods of duration and various areas, as determined for Boston and New Orleans storms, are given in Table 23.

There is some justification for the belief that, irrespective of locality, downpours of equal duration and having the same maximum intensity of precipitation at the eye or focus of the storm will cover corresponding areas with about the same average intensity. If this proves to be true, data from storms occurring in New Orleans, or other places where precipitation of high intensity is of frequent occurrence, can be used to supplement data from other parts of the country, such as those from Boston, thereby adding greatly to the value of such records.

49. Rational Method of Estimating Run-off.—Any method of estimating run-off requires that the engineer rely upon his judgment. No two engineers working independently are likely to reach precisely the same conclusions regarding the extent of the improvement of a drainage district within the

[1] *Trans.* Am. Soc. C. E., 1924; **87**, 535.

economic period for which drains should be built now, the rate of rainfall for which drainage should be provided, and the rate at which storm water will reach the sewers.

TABLE 23.—RATIOS OF AVERAGE INTENSITY OF PRECIPITATION OVER VARIOUS AREAS TO MAXIMUM WITHIN THE AREA, PERCENTAGE

(Based upon storms at Boston, Mass., and New Orleans, La.)

Area, acres	Duration, 15 min.	Duration, 30 min.	Duration, 45 min.	Duration, 60 min.
0	100	100	100	100
500	91	94	95	97
1,000	87	91	93	95
1,500	84	89	91	93
2,000	81	87	90	92
3,000	77	83	87	89
4,000	73	80	85	88
5,000	69	77	83	86

The rational method of estimating run-off has come into favor because it enables the engineer to apply judgment directly to specific allowances which are subject to analysis, measurement and estimate. It requires him to exercise judgment logically, after an analysis of the local conditions. The rational method is based on the direct relation between the rainfall and run-off expressed by

$$Q = ciA$$

Where Q = run-off in c.f.s. from the given area.
c = run-off coefficient = ratio of run-off to rainfall.
i = intensity of rainfall in c.f.s. per acre (inches per hour is the approximate equivalent).
A = drainage area in acres.

The engineer can measure A, but he must estimate the proper values of the other factors. The value of i to be used depends not only on the curves of the intensity of rainfall which fit the local conditions and the assumed period of recurrence, but also on the *time of concentration*, which is the time it takes to establish run-off and flow from the most distant point in the district to reach the nearest inlet and thence flow through the sewers to the point in the drainage system for which the required capacity is to be estimated. It is the greatest intensity of

rainfall lasting for this time of concentration for which provision must be made, not the intensity for a shorter period. The value of c must be estimated from a study of the soil, slope and character of the surface, and a consideration of probable future development.

50. Time of Concentration.—Time of concentration is made up of *inlet time*, the period required to establish run-off and for water to flow from the most distant point in the drainage area to the drain, and *time of flow* in the drain. The latter is readily estimated by hydraulic computations explained in Chapter IV. *Inlet time* is sometimes the most important factor in determining the probable run-off, particularly in small districts or in fairly large districts with steep slopes. In cities it is seldom less than 3 or more than 20 min. Horner's investigations[1] led him to conclude that along improved streets with grades of 0.5 to 5 per cent, the water from streets, sidewalks and roofs will reach the drain in 2 to 5 min., but the velocity over grass plots is so low that even in heavy rains water will take 10 to 20 min. in flowing 100 ft.

Charles E. Gregory[2] has computed theoretically the run-off during a rain of 4 in. per hour for the following cases:

Surface	Width, ft.	Length, ft.	Slope	Time required for run-off to equal rainfall rate, min.
Asphalt street	24	1,000	0.0025	42
Asphalt street	60	600	0.005	20
Roof	25	100	0.02	11

Combined sewers or drains are designed to run full, with the run-off from the roofs and streets and the flow in gutters at maximum rates, giving a minimum time of concentration which is a constant for a given sewer district in a particular state of development.

When gagings of the storm-water flow are made, the conditions are usually different from those assumed in designing, and it is the actual time of concentration which fixes the period of the rainfall with which the resulting flow must be compared. During a

[1] *Eng. News*, 1910; **64,** 326.
[2] *Trans.*, Am. Soc. C. E., 1909; **65, 393.**

moderate storm, the sewer may be but partly filled and the velocity of flow may, therefore, be considerably less than the maximum. Moreover, unless rain has previously been falling for some time, the filling of depressions and the accumulation of sufficient head to cause flow over rough or nearly flat surfaces will require an appreciable amount of time. The actual time of concentration will, therefore, exceed that of design in all cases except for a heavy rain immediately preceded by a rain sufficient to establish run-off and those which produce the same or a greater rate of run-off than that for which the sewer was designed.

51. Coefficient of Run-off.—The run-off coefficient depends upon a large number of elements and is not a constant for a given area, even during a single storm. It is seldom unity, even when the entire surface is covered with roofs and pavements, for some evaporation takes place during the storm, nominally impervious surfaces absorb some moisture, and irregularities of the surface tend to hold back some of the water. The run-off coefficient gradually increases for some time after the beginning of a rain until the soil has become thoroughly saturated, the impervious surfaces thoroughly wetted and the depressions filled. After that time the factor remains substantially constant for a given area. It therefore makes considerable difference in the rate of run-off whether the critical precipitation comes near the beginning of a storm or after rain has been falling for some time. Prof. A. J. Henry of the U. S. Weather Bureau prepared[1] Table 24 which is helpful in this connection by showing the percentage of cases of downpour in Washington, Savannah and St. Louis, in which the maximum rate of precipitation occurred within various periods after the beginning of the storm. If a warm rain falls when the surface is covered with ice, the run-off coefficient may even exceed unity.

It is evident that an exact determination of the run-off for conditions which will exist in the future is not possible.

In making an allowance for the effect of the rate of precipitation on the run-off coefficient, it must be kept in mind that in most cases the maximum rate of precipitation does not occur over a large area, and it may be proper in planning a large drainage system to use a lower average rate of precipitation than should be used for a smaller area.

[1] Bulletin D, Rainfall in the United States, U. S. Weather Bureau. *Jour.*, West. Soc. Engrs., April 1899.

In a region where the duration of rainfalls is short, the run-off coefficient may not be so high as where the rainfalls are protracted, so that the depressions in the surface become filled with water, the earth becomes saturated and the conditions are such that a larger proportion of the rainfall runs off than is the case with shorter rainfalls.

TABLE 24.—PERCENTAGE OF CASES OF EXCESSIVE RAINFALL IN WHICH THE MAXIMUM INTENSITY OF PRECIPITATION OCCURRED WITHIN VARIOUS TIME INTERVALS FROM THE BEGINNING OF THE STORM

Minutes after beginning of storm	Per cent of cases in which maximum intensity occurred within period at		
	Washington	Savannah	St. Louis
5	17	10	31
10	38	31	61
15	59	52	69
20	64	65	74
25	72	72	76
30	81	82	78
35	86	87	80
40	91	88	88
45	93	92	93
50	94	97	98
60	100	100	100

Charles E. Gregory, in "Rainfall and Run-off in Storm-water Sewers,"[1] derived a formula from his studies of some measurements by Hering of the run-off from a certain district in New York City and from published records of discharge measurements for storm-water sewers, for the relation between the run-off coefficient and the duration of the rainfall for impervious surfaces. This formula was expressed by $c = 0.175t^{1/3}$ where c is the run-off coefficient and t is the duration in minutes. The corresponding values of c for totally impervious surfaces for various times would then be

t......	3	5	10	15	20	30	45	60	90	120	180	186
c......	0.25	0.30	0.38	0.43	0.48	0.55	0.62	0.68	0.79	0.86	0.99	1.00

The duration of storms has another effect on run-off, for if the duration of a storm causing flood conditions is less than the time required for water to flow from the most distant point on the drainage area to the point for which computations are made or at which gagings are made, then the maximum discharge

[1] *Trans.*, Am. Soc. C. E., 1907; **58**, 458.

may come when less then the entire drainage area is contributing water. This condition need be considered only in preparing designs for large drainage areas, or in studying gagings of the storm flow in sewers and drains.

There is some retardation in run-off due to the time required to fill gutters and sewers and build up sufficient head to carry off the water in the drains as fast as it falls upon the tributary area. In designing sewers, it is usually best to take no account of this storage capacity, but it must be considered in studying and comparing gagings of storm flow in sewers with rainfall records.

52. Coefficient of Distribution of Rainfall.—Data upon the distribution of rainfall over a drainage area are few and inconclusive. Marston's studies, already referred to, indicate that for the smaller drainage areas this coefficient is very nearly unity for areas of 1,000 acres it may be about 0.95; for 2,500 acres, 0.90; and for 5,000 acres, 0.85, for durations of about 60 min.

53. Coefficient of Retention.—This coefficient takes account of the water required to wet the surfaces; evaporation during a storm; water held back in depressions and irregularities of the surface, by vegetation, etc., and water absorbed by porous earth, which, therefore, does not find its way into the sewers. All of these influences have vastly more effect at the beginning of a storm than after rain has been falling for some time, and also vary with climatic conditions, so that the value of this coefficient is far from constant, even for a single drainage area. Furthermore, in growing cities the extent of the areas covered by roofs and impervious pavements is continually increasing, with a corresponding diminution of pervious areas, and pavements and roofs are being made smoother and less absorbent. For this reason, present values of the coefficient are of service only for comparative purposes.

54. Coefficient of Retardation.—If the duration of the storm causing flood conditions is less than the time required to establish run-off and for water to flow from the most distant point on the drainage area to the point for which computations or gagings are made, then the maximum discharge will come when less than the whole drainage area is contributing water. The ratio of the area so contributing to the total drainage area is called the coefficient of retardation.

Obviously, if the precipitation continues at a uniform rate for an indefinite time, the greatest discharge will occur when all parts of the drainage area are contributing water, and after an interval from the beginning of the downpour equal to the time required to establish run-off and for water to flow from the most distant point (measured in time of flow) to the point under consideration. If the downpour lasts but a short time, and particularly if the drainage area is irregular in shape, it is possible that the maximum discharge may occur when but a portion of the area is contributing water. This portion will be the largest area within the drainage area and between two "contours" (lines of equal "time-distance" or equal time of flow from the point under consideration) whose distance apart, measured in time, is equal to the duration of the downpour. If this time should equal the time of concentration for the entire area, the ratio would be unity and there would be no retardation.

In problems of design in most cases the maximum discharge would result from a rain lasting for a sufficient period so that the entire area would contribute water—in other words, for a period equal to the time of concentration—and accordingly retardation usually need not be considered in such work. For large areas, particularly in problems where it is planned to build a submain to serve sub-areas which are equal to or greater than say one-half the area to be served by the main just upstream of the junction with the submain, computations should be made from a preliminary design to determine the total area which will contribute flow to the point of concentration for different time periods, somewhat shorter than the time of concentration. The discharge should then be computed for these shorter periods, using the intensity of rainfall to correspond with the shorter time, and comparison made with the discharge computed for the time of concentration of the entire area.

The coefficient of retardation should not be lost sight of in studying gagings of flow in sewers. In other words, unless it is certain that the downpour has lasted for a period equal to or exceeding the time of concentration, it must be remembered that all parts of the drainage area may not have been contributing water to the maximum discharge, and the area which was actually contributing must be determined in order to find the true run-off factor.

In making this allowance for retardation, the effect of the travel of the storm should not be overlooked. Information

on this point is usually not to be had, but would be required for a complete and accurate solution of the problem.

It must not be forgotten that the actual time of concentration for a given drainage area is not a constant and will be greater in light storms, when the drains are but partly filled, than in heavy storms, when maximum velocities are attained. This condition, like the "coefficient of retardation," is of major importance in studying gagings and comparing them with the storms producing the run-off.

55. Values of Run-off Coefficient.—There will inevitably be a difference in the influence on the run-off coefficient which different engineers attribute to each of the conditions mentioned in the last section. These differences are shown in Table 25.

Some engineers do not attempt to make close estimates of the different kinds of surface in an urban district, but content themselves with average values of the proportional run-off as follows:[1]

For the most densely built-up portion of the district	0.70–0.90
For the adjoining well built-up portions	0.50–0.70
For the residential portions with detached houses	0.25–0.50
For the suburban portions with few buildings	0.10–0.25

In general, in the absence of suitable information from which to estimate directly the run-off coefficient for a given area under conditions assumed to exist at the end of the "period of design," this factor may be most satisfactorily approximated by estimating the "equivalent percentage of totally impervious area," as it is sometimes called. Thus if it is assumed that in the future 15 per cent of the area of the district will be covered by roofs for which the coefficient would be 0.95, 30 per cent by pavements with coefficient 0.90, 40 per cent by lawns with coefficient 0.15, 15 per cent by gardens with coefficient 0.10, the resulting coefficient for the entire district will be 0.4875, or, in round numbers, 0.50.

56. Average Run-off Coefficient According to the "Zone Principle" (Time-contour Analysis).—If a drainage area be conceived as divided into zones by lines (called time contours), each connecting the points from which water will flow to the outlet in an equal time, then the water running off from any

[1] See also coefficients shown in Table 26 following.

zone at any moment will be derived from a particular part of the storm, while the run-off from the next zone will be derived from a correspondingly earlier or later portion of the storm.

Table 25.—Range in Estimates of Run-off from Different Classes of Surface in Proportions of the Rainfall Intensity

(From Bryant and Kuichling's Report on the Adequacy of the Present Sewerage System of the Back Bay District of Boston, etc., 1909)

For water-tight roof surfaces	0.70 to 0.95
For asphalt pavements in good order	0.85 to 0.90
For stone, brick and wooden block pavements with tightly cemented joints	0.75 to 0.85
For same with open or uncemented joints	0.50 to 0.70
For inferior block pavements with open joints	0.40 to 0.50
For macadamized roadways	0.25 to 0.60
For gravel roadways and walks	0.15 to 0.30
For unpaved surfaces, railroad yards and vacant lots	0.10 to 0.30
For parks, gardens, lawns and meadows, depending on surface slope and character of subsoil	0.05 to 0.25

Obviously, the time contours are likely to be very irregular, being affected by irregularities of the surface, by surface slopes, by location of inlets, by slope and length of drains, and by other factors. For practical purposes, however, it is sufficient to consider the entire drainage area as approximating a regular geometrical figure—square, rectangle, triangle, or sector—and the zones as areas of equal width between arcs of circles having a center at the outlet. A drainage area having uniform velocities of flow over the surface and in the drains would have the lowest run-off coefficient if it approximated the shape of a sector of a circle with the outlet at the center; and a triangular area with the outlet at the shortest side would have the highest run-off coefficient; the reason being that, in the case of the sector, the greater proportion of the area is farthest from the outlet and has a low coefficient of run-off due to the short duration of the rain which produces its run-off, while, in the case of the triangle, the largest zone is nearest the outlet and has also a larger run-off coefficient.

A square with the outlet at the center of a side and a rectangle with the outlet at the center of the short side would have average coefficients intermediate between those of the sector and the triangle.

Fig. 21 shows the relation between percentage of total area contributing to the flow at the outlet, and the percentage of the elapsed time (of the total time required for the entire area to contribute to the flow), for areas of the shapes and proportions shown, on the assumption of uniform velocities of flow from all portions to the outlet.

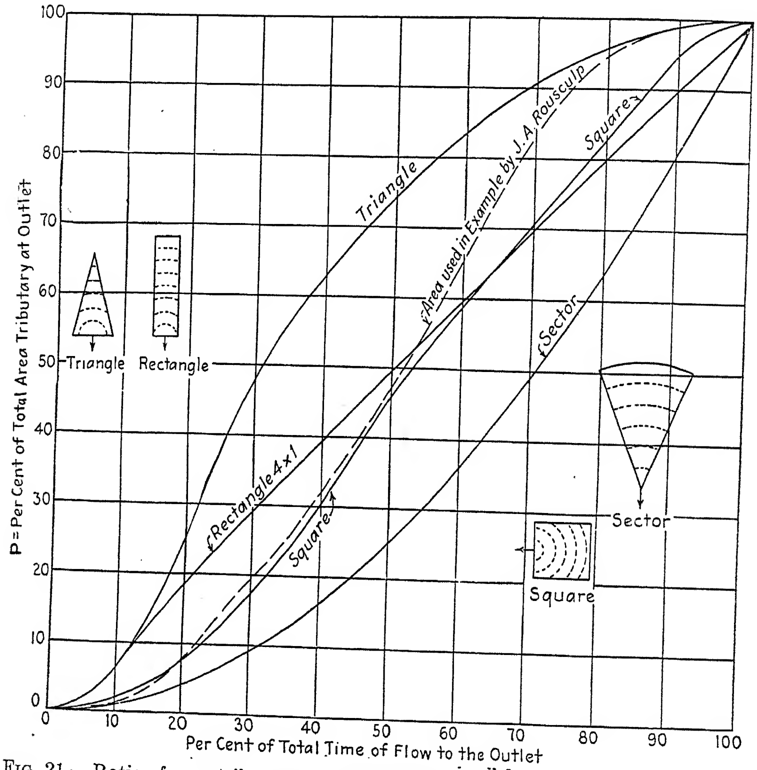


Fig. 21.—Ratio of area tributary at outlet to time of flow for regular areas with constant velocity.

Areas with varying velocities of flow, whatever their actual shape, may resemble in run-off coefficients any of the above geometric figures, depending upon the relative times of flow from the different portions to the outlet. An inspection of the areas will usually indicate the effective shapes, and applicable run-off coefficients may be chosen by the exercise of judgment.

The majority of urban drainage districts served by artificial drains will be found to approximate rectangles in effective shape.

Basic coefficients of run-off must be chosen for the drainage area under consideration, either from the results of experience or by the use of a formula, due allowance being made for the character of the soil and the degree of development. In Table 26 are shown run-off coefficients which are based upon experience and measurements of flow and have been used in some cities for several years. Experience indicates that these values are reasonably close for impervious surfaces, but those for pervious surfaces are applicable only where the soil is dense, while for sandy soils the coefficients should be smaller.

TABLE 26.—ASSUMED PERCENTAGES OF RUN-OFF (HORNER)

Duration t in minutes	Per cent run-off from	
	Impervious portion	Pervious portion
5	50	10
10	60	20
15	70	30
20	80	35
30	85	40
60	95	50
120	95	60
150	95	65
180	95	70

The basic coefficients are intended to apply to elementary areas where the time of concentration is relatively short, approximating a time commonly used as "inlet time;" thus, the run-off coefficient for a totally impervious area having a time of concentration of, say, 30 min., would not be 0.85, because at the end of 30 min., that portion of the flow which comes from the most distant point (30 min. distant) represents the first flow from a surface which had not been contributing flow prior to that time. The total discharge is made up of the summation of the discharges from the various zones in which the coefficients decrease progressively from 0.85 to 0.50 as the time from the outlet increases. The actual composite factor or average run-off coefficient for any drainage area will, therefore, depend upon the relative sizes

TABLE 27.—COMPUTATION OF AVERAGE RUN-OFF COEFFICIENTS FOR IMPERVIOUS SURFACES, FOR RECTANGULAR AREAS IN WHICH THE LENGTH IS FOUR TIMES THE BREADTH. COMPUTED FROM BASIC COEFFICIENTS IN TABLE 26

(P = percentage of area of one zone to total tributary area)

Duration of rain on zone in min.	Run-off coefficient for one zone after rainfall of duration stated C_1	Period of concentration, minutes																							
		180		150		135		120		105		90		75		60		45		30		20		10	
		P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P
5	0.50	2.8	1.4	3.3	1.6	3.7	1.8	4.2	2.1	4.8	2.4	5.6	2.8	6.7	3.4	8.4	4.2	11.2	5.6	16.8	8.4	25.2	12.6	50.4	25.2
10	0.60	2.8	1.7	3.4	2.0	3.8	2.3	4.2	2.5	4.8	2.9	5.6	3.4	6.7	4.0	8.4	5.0	11.2	6.7	16.8	10.1	25.2	15.1	49.6	29.8
15	0.70	2.8	2.0	3.3	2.3	3.7	2.6	4.2	2.9	4.8	3.4	5.6	3.9	6.7	4.7	8.4	5.9	11.2	7.8	16.8	11.8	25.6	17.9		
20	0.80	2.8	2.2	3.4	2.7	3.7	3.0	4.2	3.4	4.8	3.8	5.6	4.5	6.8	5.5	8.4	6.7	11.2	9.0	16.9	13.5	24.0	19.2		
25	0.825	2.8	2.3	3.4	2.8	3.8	3.1	4.2	3.5	4.8	3.9	5.6	4.6	6.7	5.5	8.4	6.9	11.2	9.2	17.7	14.6				
30	0.85	2.8	2.4	3.3	2.8	3.7	3.1	4.2	3.6	4.8	4.1	5.6	4.8	6.7	5.7	8.4	7.1	11.3	9.6	15.0	12.7				
35	0.867	2.8	2.4	3.4	2.9	3.7	3.2	4.2	3.6	4.8	4.2	5.6	4.9	6.8	5.9	8.4	7.3	11.6	10.1						
40	0.883	2.8	2.5	3.4	3.0	3.8	3.4	4.2	3.7	4.8	4.2	5.6	4.9	6.7	5.9	8.5	7.5	13.3	11.7						
45	0.90	2.8	2.5	3.3	3.0	3.7	3.3	4.2	3.8	4.8	4.3	5.6	5.0	6.8	6.1	8.7	7.8	7.8	7.0						
50	0.917	2.8	2.6	3.4	3.1	3.7	3.4	4.2	3.8	4.8	4.4	5.6	5.1	6.8	6.2	9.0	8.3								
55	0.933	2.8	2.6	3.4	3.2	3.8	3.5	4.2	3.9	4.8	4.5	5.6	5.2	6.8	6.3	10.6	9.9								
60	0.95													25.8	24.5	4.4	4.2								
75	0.95									47.2	44.8	38.4	36.5												
90	0.95					58.9	56.0	53.8	51.1																
105	0.95	69.2	65.7	63.0	59.8																				
120	0.95																								
135	0.95																								
150	0.95																								
180	0.95																								
Average coefficient in per cent = ΣC_1P		90.3		89.2		88.7		87.9		86.9		85.6		83.7		80.8		76.7		71.1		64.8		55.0	

of the zones and the relative proportion of impervious and pervious areas in each zone, and these conditions may vary widely.

Starting with certain basic coefficients for elementary areas, such as those given in Table 26, the computation of the corresponding average coefficients of run-off according to the "zone principle" for any regular figure is simple. Details of such computations for a rectangular area having a length equal to four times the breadth and with outlet at the center of the short side are shown in Tables 27 and 28, applicable to impervious areas and pervious areas having clayey soil, respectively.

The method of finding the average run-off coefficient is as follows:

From Fig. 21 find the different percentages of the area tributary to the outlet for different percentages of the total time of flow or *period of concentration*. Thus for a rectangular area having a 20-min. period of concentration the percentages contributing in 25, 50, 75 and 100 per cent of the period of concentration, are 24.0, 49.6, 74.8 and 100, respectively, and the differences between these percentages represent the areas of the individual 5-min. zones. These areas, therefore, beginning with that nearest the point of concentration, are 24.0, 25.6, 25.2 and 25.2 per cent of the total area. These are the values of P in Tables 27 and 28 for a 20-min. period of concentration, and when multiplied by the corresponding run-off coefficients give the percentages of the rainfall on the whole area which will be contributed by each zone. The summation of these percentage gives the average run-off coefficient for the area.

Similar tables should be prepared for a sufficient number of regular areas to provide a suitable basis for comparison, from which the designer may select figures applicable to the actual drainage areas under consideration.

Having made a decision as to the form and proportions of the equivalent regular area to be used and adopted the corresponding average coefficients of run-off for pervious and impervious surfaces, these may be combined in the ratio of the corresponding areas as in Table 29. Column 00 represents values from Table 28 while column 100 is taken similarly from Table 27. The remaining columns are computed from these two in proportion to the per cent of impervious surface indicated.

As has been previously stated, most urban drainage areas approximate a rectangle in effective shape. It will be apparent

TABLE 28.—COMPUTATION OF AVERAGE COEFFICIENTS FOR PERVIOUS SURFACES, ON CLAYEY SOILS, FOR RECTANGULAR AREAS IN WHICH THE LENGTH IS FOUR TIMES THE BREADTH. COMPUTED FROM BASIC COEFFICIENTS IN TABLE 26

(P = the percentage of area of one zone to the total tributary area)

Duration of rain on zone in min.	Run-off coefficient for one zone after rainfall of duration stated C_1	Period of concentration-minutes																							
		180		150		135		120		105		90		75		60		45		30		20		10	
		P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P
5	0.10	2.8	0.3	3.3	0.3	3.7	0.4	4.2	0.4	4.8	0.5	5.6	0.6	6.7	0.7	8.4	0.8	11.2	1.1	16.8	1.7	25.2	2.5	50.4	5.0
10	0.20	2.8	0.6	3.4	0.7	3.8	0.8	4.2	0.8	4.8	1.0	5.6	1.1	6.7	1.3	8.4	1.7	11.2	2.2	16.8	3.4	25.2	5.0	49.6	9.9
15	0.30	2.8	0.8	3.3	1.0	3.7	1.1	4.2	1.3	4.8	1.4	5.6	1.7	6.7	2.0	8.4	2.5	11.2	3.4	16.8	5.0	25.6	7.7		
20	0.35	2.8	1.0	3.4	1.2	3.7	1.3	4.2	1.5	4.8	1.7	5.6	2.0	6.8	2.4	8.4	2.9	11.2	3.9	16.9	5.9	24.0	8.4		
25	0.375	2.8	1.0	3.4	1.3	3.8	1.4	4.2	1.6	4.8	1.8	5.6	2.1	6.7	2.5	8.4	3.2	11.2	4.2	17.7	6.6				
30	0.48	2.8	1.1	3.3	1.3	3.7	1.5	4.2	1.7	4.8	1.9	5.6	2.2	6.7	2.7	8.4	3.4	11.3	4.5	15.0	6.0				
35	0.417	2.8	1.2	3.4	1.4	3.7	1.5	4.2	1.8	4.8	2.0	5.6	2.3	6.8	2.8	8.4	3.5	11.6	4.8						
40	0.435	2.8	1.2	3.4	1.5	3.8	1.7	4.2	1.8	4.8	2.1	5.6	2.4	6.7	2.9	8.5	3.7	13.3	5.8						
45	0.45	2.8	1.3	3.3	1.5	3.7	1.7	4.2	1.9	4.8	2.2	5.6	2.5	6.8	3.0	8.7	3.9	7.8	3.5						
50	0.467	2.8	1.3	3.4	1.6	3.7	1.7	4.2	2.0	4.8	2.3	5.6	2.6	6.8	3.2	9.0	4.2								
55	0.483	2.8	1.3	3.4	1.6	3.8	1.8	4.2	2.0	4.8	2.3	5.6	2.7	6.8	3.3	10.6	5.1								
60	0.50	2.8	1.4	3.3	1.7	3.7	1.8	4.2	2.1	4.8	2.4	5.7	2.8	7.0	3.5	4.4	2.2								
65	0.508	2.8	1.4	3.4	1.7	3.7	1.9	4.2	2.1	4.8	2.4	5.8	3.0	7.8	3.9										
70	0.517	2.8	1.4	3.4	1.8	3.8	2.0	4.2	2.2	4.9	2.5	5.8	3.0	8.2	4.2										
75	0.525	2.8	1.5	3.3	1.7	3.7	1.9	4.2	2.2	5.0	2.6	6.1	3.2	2.8	1.4										
80	0.533	2.8	1.5	3.4	1.8	3.7	2.0	4.3	2.3	5.0	2.7	7.2	3.8												
85	0.542	2.8	1.5	3.4	1.8	3.8	2.1	4.3	2.3	5.1	2.8	5.8	3.1												
90	0.55	2.8	1.5	3.4	1.9	3.8	2.1	4.4	2.1	5.4	3.0	2.0	1.1												
95	0.558	2.8	1.6	3.4	1.9	3.8	2.1	4.5	2.5	6.5	3.6														
100	0.567	2.8	1.6	3.4	1.9	3.9	2.2	4.5	2.6	4.3	2.4														
105	0.575	2.8	1.6	3.4	2.0	3.9	2.2	5.1	2.9	1.4	0.8														

110	0.583	2.8	1.6	3.4	2.0	4.0	2.3	5.5	3.2																
115	0.592	2.8	1.7	3.4	2.0	4.2	2.5	3.3	2.0																
120	0.600	2.9	1.7	3.6	2.2	5.1	3.1	1.1	0.7																
125	0.608	2.9	1.8	3.8	2.3	4.3	2.6																		
130	0.617	2.9	1.8	4.0	2.5	2.6	1.6																		
135	0.625	2.9	1.8	4.7	2.9	0.9	0.6																		
140	0.633	2.9	1.8	3.5	2.2																				
145	0.642	3.0	1.9	2.1	1.4																				
150	0.650	3.1	2.0	0.7	0.5																				
155	0.658	3.3	2.2																						
160	0.667	3.9	2.6																						
165	0.675	3.4	2.3																						
170	0.683	2.4	1.6																						
175	0.692	1.5	1.0																						
180	0.700	0.5	0.3																						
Average coefficient in per cent = $\Sigma C_1 P$..........		52.2		49.5		47.9		46.3		44.5		42.2		39.8		37.1		33.4		28.6		23.6		14.9	

from a study of Fig. 21 that run-off coefficients do not vary widely for rectangular areas with different ratios of length to width. In the case which has been worked out where the length equals four times the width, for a duration of rainfall of 60 minutes, the run-off coefficient is 80.8 for impervious surfaces, while for a square it would be 81.1. This difference is less than

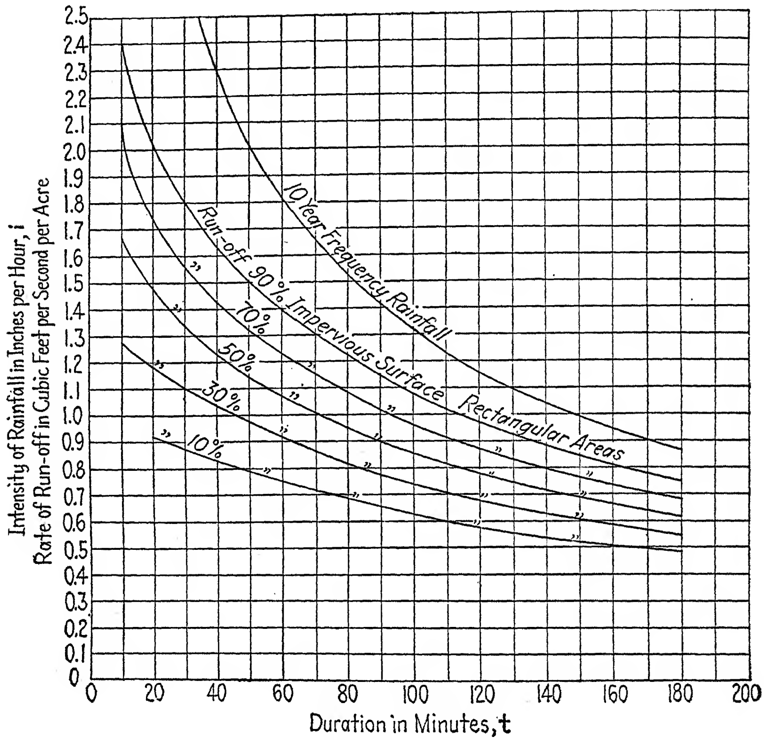


FIG. 22.—Runoff curves for Detroit, Mich., based upon rainfall curve of 10-year frequency and various percentages of impervious surfaces and clay soil.

Computed for rectangular area with length = 4 × breath, from basic coefficients in Table 26, combined according to "zone principle."

the probable errors in assumptions so that the coefficients given in Table 29 should be applicable to most drainage areas with a reasonable degree of accuracy. Since the curve for the 4 by 1 rectangle used almost coincides with the diagonal, which also represents an approximate mean for the various geometrical shapes, it is suggested that this diagonal might be used with equal accuracy and greater ease in computing.

The zone principle is thus seen to be a method of evaluating more accurately the variables affecting the selection of coefficients of run-off. The run-off is then determined by applying these coefficients to the rainfall curve and proceeding by the rational method already described. In Fig. 22 run-off curves are plotted for Detroit, Michigan, showing the application of the zone principle to a specific case by applying the coefficients in Table 29 to the 10-year rainfall intensity duration curve.

Table 29.—Runoff Coefficients for Rectangular Areas in Which the Length is Four Times the Breadth, Containing Various Percentages of Impervious Surfaces; Computed from the Basic Coefficients Given in Table 26, Combined According to the Zone Principle

Duration, min. or time of concentration, t	Per cent of impervious surfaces										
	00	10	20	30	40	50	60	70	80	90	100
10	0.149	0.189	0.229	0.269	0.309	0.350	0.390	0.430	0.470	0.510	0.550
20	0.236	0.277	0.318	0.360	0.401	0.442	0.483	0.524	0.566	0.607	0.648
30	0.287	0.329	0.372	0.414	0.457	0.499	0.541	0.584	0.626	0.669	0.711
45	0 334	0.377	0.421	0.464	0.508	0.551	0.594	0.638	0.681	0.725	0.768
60	0.371	0.415	0.458	0.502	0.546	0.590	0.633	0.677	0.721	0.764	0.808
75	0.398	0.442	0.486	0.530	0.574	0.618	0.661	0.705	0.749	0.793	0.837
90	0.422	0.465	0.509	0.552	0.596	0.639	0.682	0.726	0.769	0.813	0.856
105	0.445	0.487	0.530	0.572	0.615	0.657	0.699	0.742	0.784	0.827	0.869
120	0.463	0.505	0.546	0.588	0.629	0.671	0.713	0.754	0.796	0.837	0.879
135	0.479	0.521	0.561	0.601	0.642	0.683	0.724	0.765	0.805	0.846	0.887
150	0.495	0.535	0.574	0.614	0.654	0.694	0.733	0.773	0.813	0.852	0.892
180	0.522	0.560	0.598	0.636	0.674	0.713	0.751	0.789	0.827	0.865	0.903

57. Empirical Methods for Estimating Storm-water Flow.—In the earlier plans for drains and channels to carry away the water of storms, engineers based their designs largely upon their observations of the volumes of water seen coming from known areas in times of storm and upon the sizes of natural gutters or water courses with which they were more or less familiar. Later, the tributary areas, which could be accurately measured, were introduced as constants, and the estimates of run-off were based upon a given depth of precipitation over the whole district; but with further study it was learned that there is a gradual reduction in the immediate run-off per acre with an increase in the extent of the area and, accordingly, formulas were devised by which this fact was ta into account more or less empirically. Still more recently it has been recognized that

differences in the rainfall, and especially in the intensity of the precipitation, have a direct influence upon the resulting storm-water flow, and other factors have been introduced into the formulas to take account of this and of the slope and dimensions of the drainage area. The result has been the gradual development of a number of empirical formulas and diagrams, by which the greatest quantity of storm water to be discharged from any given drainage area could be estimated.

58. Empirical Formulas.—The best known of these empirical formulas, reduced to a uniform notation, and with the introduction of a term expressing rate of rainfall (which was not originally used in all of them), are as follows:

Hawksley (London, 1857): $Q = Aci\sqrt[4]{(S/Ai)}$, in which $c = 0.7$ and $i = 1.0$

Bürkli-Ziegler (Zurich, 1880): $Q = Aci\sqrt[4]{(S/A)}$, in which $c = 0.7$ to 0.9, and $i = 1$ to 3.

Adams (Brooklyn, 1880): $Q = Aci\sqrt[12]{(S/A^2i^2)}$, in which $c = 1.035$ and $i = 1$.

McMath (St. Louis, 1887): $Q = Aci\sqrt[5]{(S/A)}$, in which $c = 0.75$ and $i = 2.75$

Hering (New York, 1889) $Q = ciA^{0.85}S^{0.27}$, or $Q = Aci\sqrt[6]{(S^{1.62}/A)} = ciA^{0.833}S^{0.27}$, in which ci varies from 1.02 to 1.64. These two formulas give considerably different results.

Parmley (Cleveland, 1898) $Q = Aci\sqrt[6]{(S^{1.5}/A)}$, in which c is between 0 and 1, and $i = 4$.[1]

Gregory (New York, 1907) $Q = AciS^{0.186}/A^{0.14}$, in which $ci = 2.8$ for impervious surfaces, and S = average slope of the surface of the ground, in feet per thousand.

A comparison of these formulas is shown in Fig. 23, for a slope of 10 ft. per 1,000 ft. It will be seen that a very wide range of results may be obtained, depending upon the formula chosen.

Of these, the Bürkli-Ziegler and McMath formulas are still used to some extent. The others are of historical interest only, except as they may be needed for reference in connection with sewers and drains built in earlier times.

[1] Parmley takes i as representing the intensity of rainfall for a period of 8 or 10 min., and for the Walworth Run Sewer (Cleveland) used $i = 4$ in order to provide for the most violent storms, and also for the further damage caused by the prevailing direction of the storms.

It should be remembered that a run-off equivalent to 1 in. in depth in 1 hour from an area of 1 acre equals 1.008 cu. ft. per second.

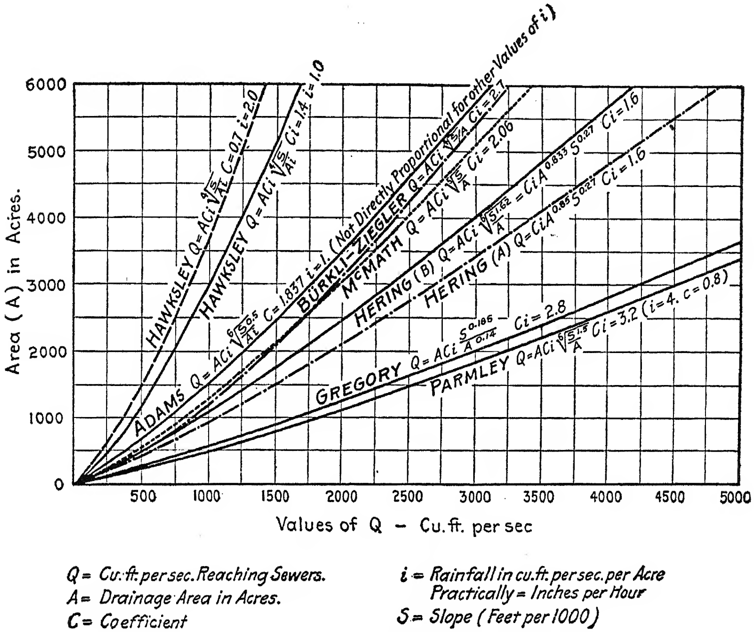


FIG. 23.—Comparison of run-off formulas for slope, S, of 10 ft. in 1,000 ft. and large areas.

59. The Use of McMath's Formula.—Of the foregoing formulas, that of McMath is probably most favorably known, and it has been widely used; often, no doubt, without careful study into its applicability. While the use of this or any similar formula is not to be recommended when sufficient information is available for the application of the rational method, yet there are cases when its use may be warranted. It is also convenient for use in rough preliminary computations, as it can be employed very rapidly by means of tables or diagrams with sufficient accuracy for such purposes, and indeed, with greater precision than the applicability of the formula warrants.

Convenient diagrams for the solution of McMath's formula are given in Figs. 24, 25 and 26. In using these diagrams, start with the given area at the bottom of the diagram and follow a

vertical line to its intersection with the slope line; then follow a horizontal line to its intersection with the *ci* line (having first found from Table 30 or by multiplication, the product of the assumed coefficient of run-off *c* and the intensity of precipitation

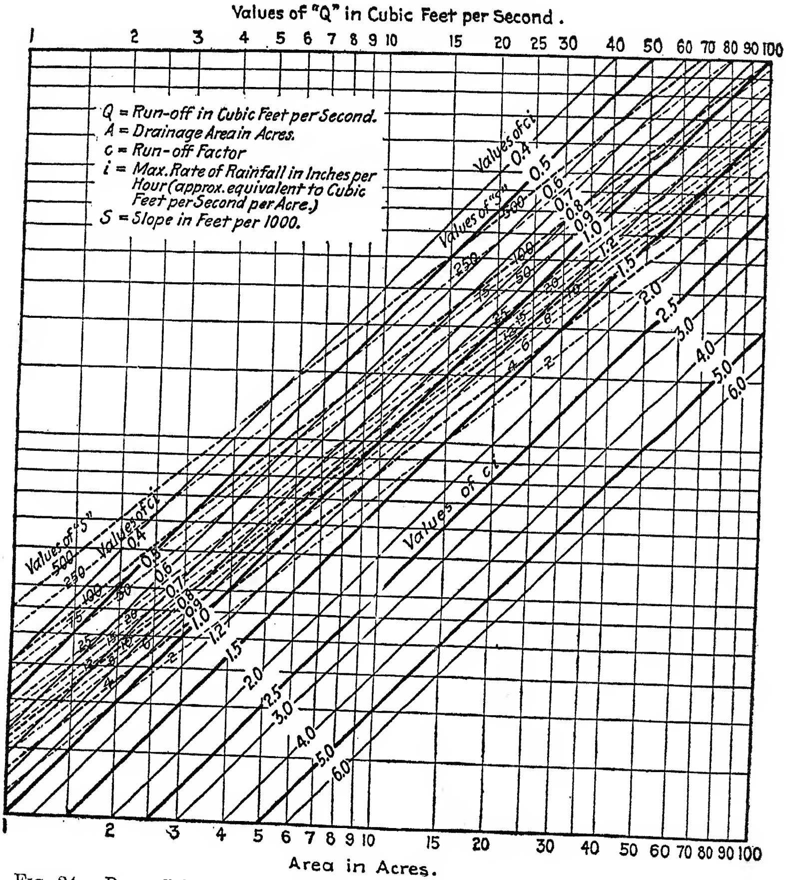


FIG. 24.—Run-off from sewered areas of 1 to 100 acres, by McMath's formula.

i); from this point follow a vertical line to the scale of quantities at the top of the diagram. For example, assume $A = 100$ acres, $i = 3$ in., $c = 0.70$, and $S = 15$. Then $Q = 144$ cu. ft. per second.

The values of *ci* for use with these diagrams are given in Table 30.

TABLE 30.—VALUES OF *ci* FOR USE WITH FIGS. 24, 25, AND 26

c	*i*					
	2.25	2.50	2.75	3.00	3.50	4.00
0.3	0.68	0.75	0.83	0.90	1.05	1.20
0.4	0.90	1.00	1.10	1.20	1.40	1.60
0.5	1.13	1.25	1.38	1.50	1.75	2.00
0.6	1.35	1.50	1.65	1.80	2.10	2.40
0.7	1.58	1.75	1.93	2.10	2.45	2.80
0.75	1.69	1.88	2.06	2.25	2.63	3.00
0.8	1.80	2.00	2.20	2.40	2.80	3.20
0.9	2.03	2.25	2.48	2.70	3.15	3.60

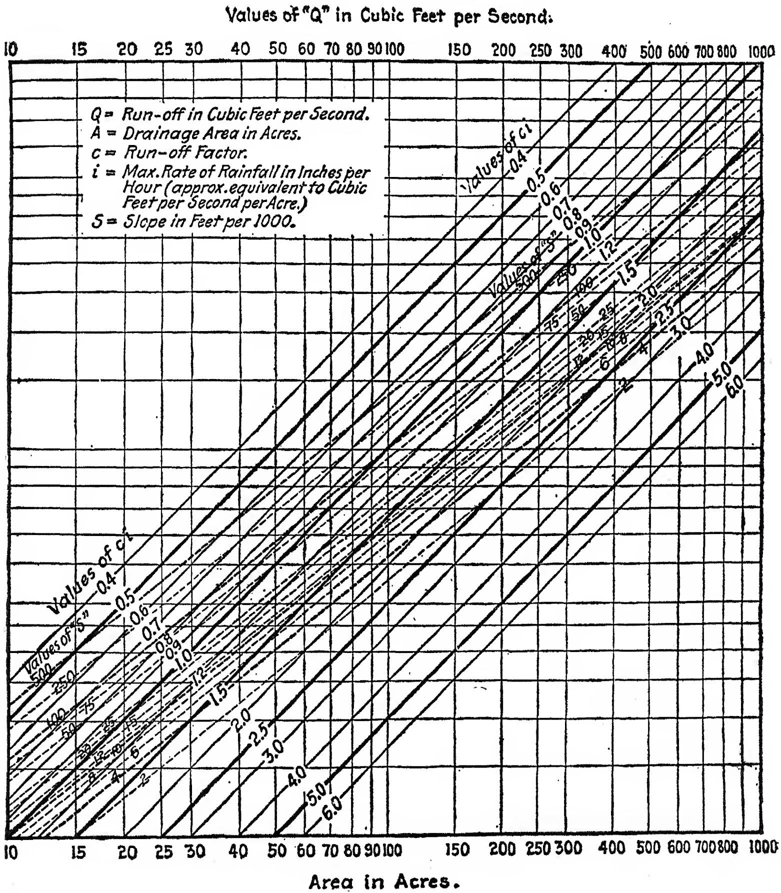


FIG. 25.—Run-off from sewered areas of 10 to 1,000 acres, by McMath's formula.

Values of c suggested[1] for various proportions of impervious surface are:

Impervious area, per cent									
Sandy soil..........	0	5	10	16	25	37	53	73	100
Clayey soil..........				5	15	28	46	70	100
Value of c...........	0.10	0.14	0.18	0.23	0 30	0.40	0.50	0.70	0.90

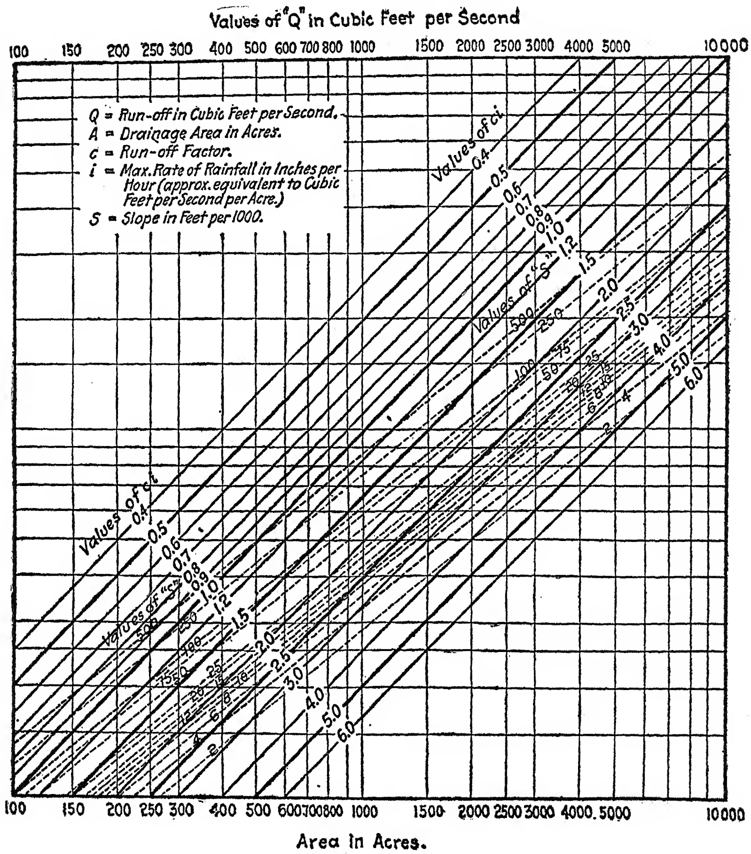


FIG. 26.—Run-off from sewered areas of 100 to 10,000 acres, by McMath's formula.

63. Flood Flows from Large Areas.—The foregoing discussion relates to the estimation of storm run-off for the design of storm-water drains and combined sewers. The empirical formulas have all been derived from observations of run-off from sewered

[1] Am. C. E. Handbook, 4th ed., p. 1254.

areas of comparatively small extent, and their use for large areas is not justified by their source.

It sometimes becomes necessary, in drainage problems, to consider comparatively large areas, especially in cases where a stream passing through a city is to be carried in a covered channel. Formulas sometimes used for estimating flood flows from large areas are plotted in Fig. 27 together with some indication of the probable frequency of such flows.

61. Application of the "Rational Method" to Large Drainage Areas.—Logically, the rational method of estimating storm run-off is as applicable to large as to small drainage areas. Practically, its application to large areas is less certain than to small ones, because of the uneven distribution of precipitation, the uncertainty as to rainfall over the entire area, the lack of definite knowledge of time of concentration, run-off coefficient, and the like. Actual records of flood flows of the stream under consideration, if available, are better than any estimation can be, provided the records cover a sufficient period of time.

It may often happen, however, that much more satisfactory information relative to the intensity and frequency of excessive rains can be obtained, than to any other conditions bearing upon magnitude and frequency of flood flows. In such cases, it may be possible to make a better estimate by the rational method than in any other way. The procedure in this case is similar to that used for small areas.

Coefficients of run-off for extreme floods are likely to be high, because such floods often come when the ground is frozen or sheathed with ice, or, in some places, when a torrential rain falls on sun-baked and largely impervious soil. The coefficient for the Scioto River (Ohio) flood of 1913, with a drainage area of 1,050 square miles and a period of concentration of about 30 hours, was found to be 0.68 for the maximum 72 hours, 0.66 for the maximum 48 hours, and 0.59 for the maximum 24 hours. The ground was water-soaked and practically frozen, and other conditions were favorable for maximum run-off. In the same storm the Miami River (Ohio), with drainage area 2,525 square miles, showed coefficients of 0.91, 0.92, and 0.76 for 72, 48 and 24 hours, respectively. The period of concentration was about 48 hours.

In estimating the frequency of flood flows of streams, it must be remembered that the simultaneous occurrence of excessive

rainfall and conditions causing maximum run-off coefficient is necessary to cause a maximum flood flow and that a combination of these conditions occurs very infrequently.

62. Relative Adequacy of Laterals and Mains.—Much more damage will generally result from lack of capacity of a main drain than from the inadequacy of one submain or lateral; yet in flat low-lying areas inadequate laterals may result in serious damage. In general, laterals and submains are built in the portions of a district which have the ground surface at higher elevations than in areas adjacent to the main, and flood water which cannot enter the laterals and submains will find its way over the surface to the low areas where damage may result. It is simpler and usually less expensive to reinforce a main sewer or drain than to rebuild many small laterals. The additional cost of constructing the latter of ample size when first built will generally be inconsiderable, while the additional cost may prohibit the construction of a main sewer large enough to care for the run-off from the most severe storms many years hence, after the ground area has been made much more impervious by pavements, sidewalks and many more buildings. It is generally advisable, therefore, to build lateral sewers large enough for the ultimate requirements, giving the mains and submains a sufficient capacity for the probable run-off during a comparatively short term, with the expectation that relief sewers will eventually be built to care adequately for the entire run-off from the district.

63. Sewer Gaging.—It is rarely practicable to gage the flow of storm water in sewers by weirs, current meters and other devices employed in open channels. Generally the depth of the sewage is observed at several places and from such data and the slope and an estimate of the roughness of the sewers between these places the approximate discharge is computed by the methods explained in Chapter IV. Storms may occur at many times when observers cannot be stationed in the sewers to measure the depth of the storm water, so automatic recording gages are desirable. At least two are necessary to determine the slope of the water surface, which frequently differs from the slope of the sewer.

The City of New York in 1926 installed a large Venturi flume in one of the sewers in Manhattan Borough. It is probable that this will furnish more accurate results than have previously been obtained in storm-water gagings; but in some

cases the obstruction to flow caused by such a flume or any other device would not be allowable.

64. Unsatisfactory Nature of Sewer Gagings.—It must be admitted that the determination of the run-off factor from actual gagings is extremely unsatisfactory. Only a limited number of such gagings have been made and even the best of these leave much to be desired; the coefficients deduced from them can be considered only as approximations. Nevertheless, these measurements are of much importance, not only because they furnish the only experimental determinations of the run-off factor which are to be had, but because a careful study of them aids materially in training the judgment and in arriving at a clear and full conception of the problem.

For an exact analysis of the relation between precipitation and run-off, it is necessary to know the true rainfall upon the district drained, including the distribution of rainfall over the entire area at all times during the storm, and the true storm run-off, including not only the quantity flowing past the gaging point at all times during and immediately before and after the storm, but the amount which could have been concentrated at this point if the conditions had been favorable. For instance, if the critical precipitation comes at the beginning of a storm when the flow in the sewers is small and the velocity of flow slight, a very considerable portion of the run-off from the surface will be required to fill the sewers. In this case, the velocity of flow will be small, the time of concentration will be long, and the actual maximum rate of flow in the sewer will be materially less than the real rate of run-off. If, on the other hand, the critical precipitation occurs after a long period of moderately heavy rain, particularly if accompanied by melting snow, when the storage space in the sewers is largely filled and the velocity of flow is at a maximum, the quantity actually flowing in the sewer will represent very nearly the true run-off from the storm if the sewers do not become surcharged, and the time of concentration will be a minimum. Whether or not the storm-water inlets are adequate to admit water into the sewer as rapidly as it reaches them is also of importance.

65. Gages.—The float gage used in recording the water level in a sewer is actuated by a float in a pipe or other suitable guide in which the sewage stands at the same height as in the sewer. A cord, chain, tape or rod runs from the float to the

recording apparatus, of which there are several types. All are driven by clockwork and the clock movement should be regulated to keep correct time and be synchronized with the clocks of all other gages furnishing records to be studied jointly.

An apparatus of this type made by the Builders Iron Foundry is shown in Fig. 28. A cord from the float moves an arm carrying a pen in front of a circular chart rotated by clockwork. The pen moves in a circular arc, and consequently the time-scale

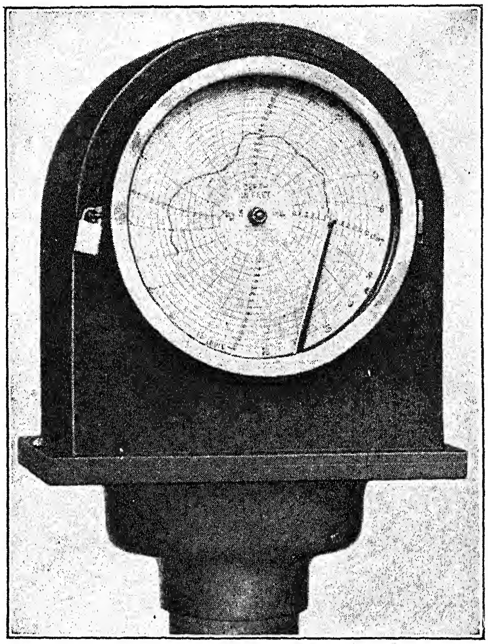


FIG. 28.—Water-level recorder (Builders Iron Foundry).

is unduly small when the pen is near the center of the chart. The instrument is enclosed in a cast-iron box mounted on a hollow standard through which the float cord passes. It is made in two sizes, with 8- and 12-in. dials. The scale of heights as recorded on the chart will depend upon the range to be covered and the size of the chart.

The pneumatic type of pressure gage has a diaphragm box or pressure chamber which is placed in the liquid. The rise and fall of the surface of the liquid varies the pressure within

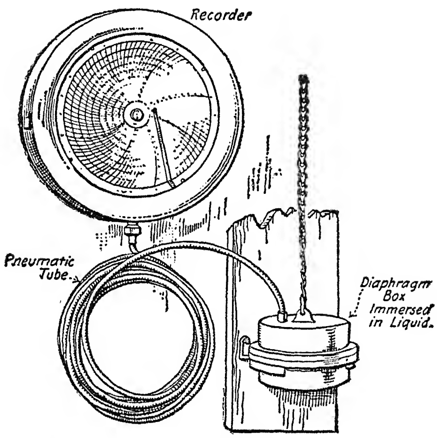


FIG. 29.—Diaphragm pressure gage.

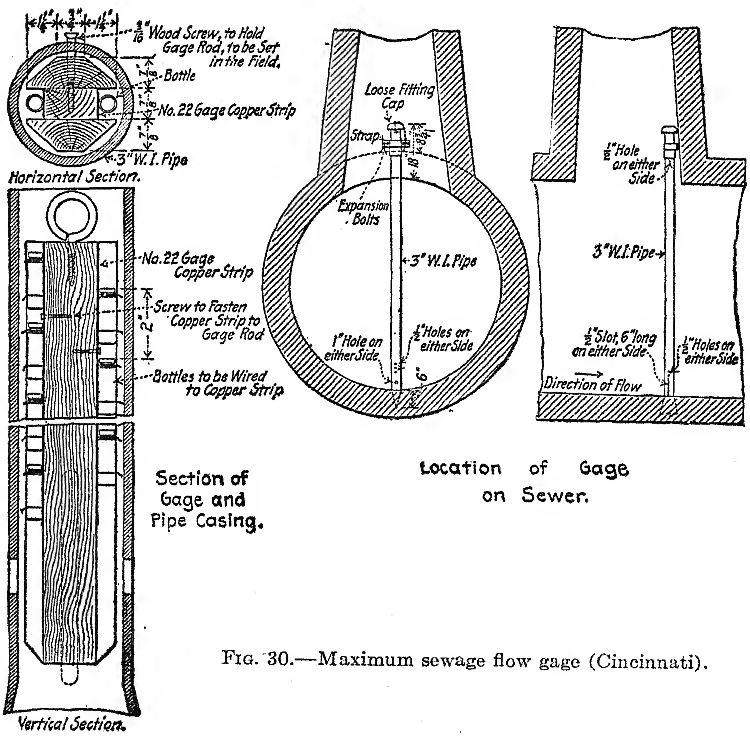


FIG. 30.—Maximum sewage flow gage (Cincinnati).

the box, and these variations are transmitted through a small air pipe to the recording apparatus. Fig. 29 shows an instrument of this type made by The Foxboro Company and The Bristol Company. It is made for either 8- or 12-in. charts.

A type of maximum flow gage devised at Cincinnati under the direction of H. S. Morse is shown in Fig. 30. It consists of a wooden staff held firmly in place inside a vertical steel pipe. Attached to the staff are bottles arranged so their mouths are 1 in. apart vertically. There are perforations in the steel pipe through which sewage enters and rises alongside the staff, so that the highest bottle filled with sewage shows that the sewer has been filled to at least that height. After a storm, the staff is lifted out of the pipe, the maximum flow elevation ascertained from the highest filled bottle, the staff is reversed to empty the bottles and then set in place in the pipe ready for service again. This gage has proved satisfactory under ordinary conditions but not with velocities exceeding 8 ft. per second.

66. Setting Sewer Gages.—The gage must be placed so that the condition of the sewer for a considerable distance upstream and a less distance downstream permits the discharge to be computed from the depth. The cross-section and slope must be uniform, there must be no curves, inlets or obstructions to disturb the flow, the velocity of flow should not be great, and there should be no doubt about the coefficient of roughness to use in the formula for computing the flow. In order to be sure of the results, it is necessary to have gages at each end of such a stretch of sewer to determine the slope of the water surface.

The gaging apparatus should be installed in a separate chamber or recess in a manhole so as to be protected and also easily accessible. The chamber should be connected with the sewer so that the elevation of the sewage will be the same in it and in the sewer. It is desirable to have a small flow of clean water through the gaging chamber so as to have the liquid about the apparatus or floats free from matter that might clog or derange them and become offensive through decomposition. Fig. 31 shows such a gaging chamber built at Cincinnati for holding a pneumatic-type pressure gage. The air pipe runs to an iron box at the curb to hold the recording apparatus, or the recorder can be set within a house. If a float-actuated gage is used, there is not so much range of choice for the location of the recording apparatus, but it should not be placed in the chamber if this

can be avoided, because the moisture there affects the delicate metal parts of the mechanism and the paper of the chart.

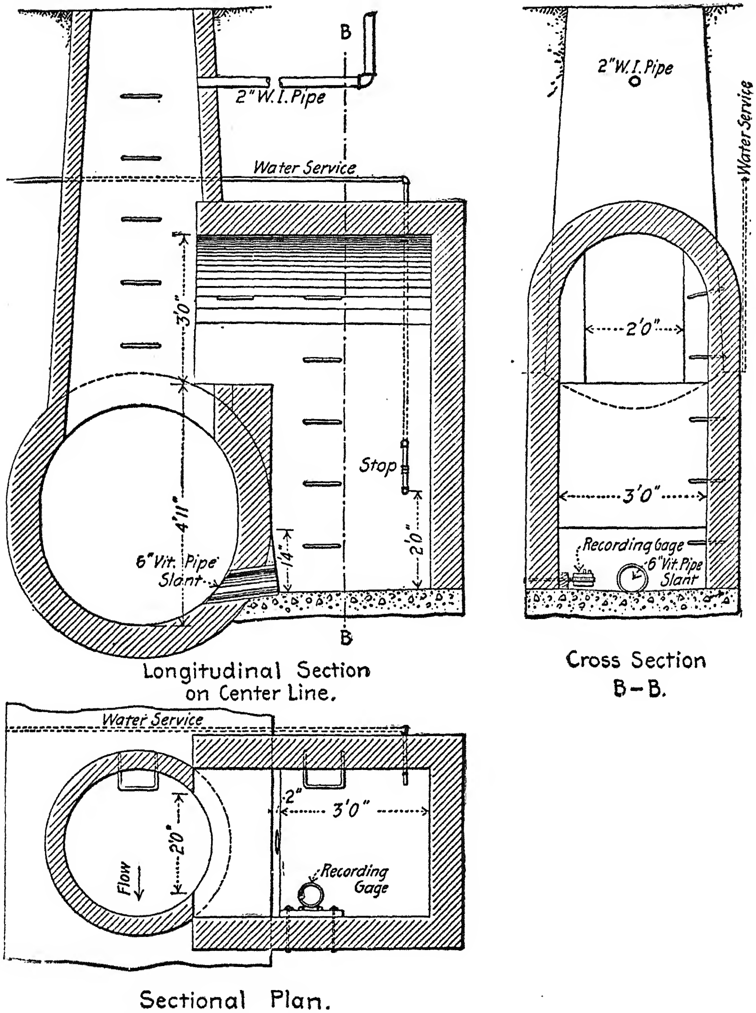


FIG. 31.—Gaging chamber, Cincinnati sewerage system.

Problems

1. Determine the average intensities of precipitation for the maximum 15; 30; 45· 60; 75; 90; 105 and 120 minute periods of the storm of August 18, 1879, from the records shown in Fig. 16.

2. What capacity should be provided in a drain to serve a tributary area of 1,800 acres with 50 per cent of the surface impervious, the time of concentration 80 min., and other data of design as shown in Fig. 22?

3. If the area in the preceding problem has such characteristics that the run-off from 1,400 acres, with an average of 60 per cent impervious surface, will be concentrated at the outlet in 50 min.; and from 1,600 acres, with 55 per cent of impervious surface, in 55 min.; what capacity should be provided in the drain?

4. Determine the required capacity for the Cambridge Street drain, Fig. 54, p. 178, just north of manhole 72, using the run-off curves given in Fig. 22, 40 per cent impervious surface and period of concentration 25 min.

5. In Table 28, c for pervious surface and 60 min. concentration period is 37.1. From Fig. 21 compute c for pervious portions of a square area having 60 min. concentration period.

6. What capacity should be provided for the two areas in Problem 5 if they each contain 800 acres of 0 per cent impervious surface; percentages of run-off as in Table 26 and rainfall intensities as in Fig 20.

7. Compute the capacity required for the Cambridge Street drain in Problem 2 using McMath's formula with $c = 0.4$, $i = 3.0$ and $S = 8.0$.

CHAPTER IV

HYDRAULICS OF SEWERS

67. Sewage is composed of about 99.9 per cent of water and but 0.1 per cent of mineral and organic matter[1] and has a specific gravity but very little in excess of unity (1.001, approximately); it is treated in hydraulic discussions as if it were clear water. The retarding effects of its contents at times and under certain conditions, more particularly at the dead ends of the collecting system, are not to be lost sight of, however.

68. Flow of Water in Pipes and Conduits.—Water seeks its own level under the influence of gravity, the level or surface being approximately perpendicular to the direction of the force of gravity. If its surface be not level it will tend to flow from the higher elevation to the lower. In addition, in bodies of water of appreciable size, flow such as that of tides takes place under the influence of celestial bodies. Convection currents are set up where differences in temperature exist and flow is caused by friction of winds on the surface. In sewer design interest is confined exclusively to the flow which takes place because of difference in level or pressure, or "head" as it is called technically.

If there be available a certain difference in level—called "fall" if measured from the upper point to the lower, or "head" if measured from the lower to the upper—between two points along a pipe, conduit, or channel carrying water or any other liquid, flow will be induced at a rate dependent, first, upon the fall as compared with the length traversed; second, upon the cross-section of the pipe, conduit, or channel; third, upon the character of its interior surface; fourth, upon the condition of flow with reference to the pipe, *i.e.*, whether the pipe is under pressure or not, whether it is the water full or partly full, and whether is flowing uniformly, steadily, variably, or intermittently on account of constant or variable cross-section, or other cause; fifth, upon the presence or absence of curves or other partial obstructions, and upon the movement of air in partly filled pipes,

[1] This corresponds roughly to 1,000 parts per million of total solids

or effect of wind in the case of open channels; and, sixth, upon the character, specific gravity, and viscosity of the liquid.

Bernoulli's theorem, that the total energy in a steady stream of frictionless fluid is constant and is dependent upon the elevation plus the pressure head, may be expressed by the following formula:

$$H = h_e + h_v + h_p = h_e + \frac{v^2}{2g} + \frac{p}{w}$$

Practically, the conditions of the perfect fluid do not exist. Other elements enter the problem—the frictional resistance of the pipe, channel, or conduit to the flowing fluid, and other resistances to flow. These factors are covered in Bernoulli's theorem by the addition of other terms in the equation just given. As applied to two different points, A and B, between which there are no losses except that due to friction, using the same nomenclature and h_f being the loss in head due to the frictional resistance of the surface traversed by the fluid in passing from point A to point B, the formula may be stated:

$$H = h_e + h_v + h_p = h_e' + h_v' + h_p' + h_f$$

69. Resistances to Flow.—The more important elements of loss of head in conduits are the loss due to friction upon the interior surface of the conduit, the loss on entrance (called the "entry head"), and losses due to sudden enlargement, sudden contraction, changes in direction, partial obstructions, entrance of branches, etc. When the velocities are low, as is usually the case in sewers, losses other than that due to friction are generally so small that they may be neglected, but in some cases, particularly when high velocities are involved, the head required to produce velocity and losses from other causes may become of major importance. In long lines, the frictional loss is generally so much greater than the sum of all the others that it obscures the latter, and closely approximate computations can frequently be made as though no other losses existed.

70. Hydraulic Grade Line.—The hydraulic grade line is a line connecting the points to which water would rise at various places along any pipe or conduit, were piezometer tubes, or vertical pipes open to the atmosphere, inserted in the liquid. It is a measure of the pressure head available at these points. The hydraulic grade line will, of course, be influenced not only by the frictional resistance due to the rugosity of the surface,

but also by anything affecting the velocity head. In the case of a canal or open channel, in contradistinction to a pipe under pressure, the hydraulic grade line corresponds with the profile of its water surface.

71. Steady and Uniform Flow.—Steady flow exists in a conduit or stream when equal quantities of water pass the same point in like intervals of time, or, in other words, when the discharge is constant for equal intervals of time.

Uniform flow exists when the cross-section and the mean velocity of the flowing stream are the same at every point. Uniform flow is a steady flow in which the cross-sections of the stream are all alike, and its surface is parallel to its bed.

The difference in these two conditions of flow must be clearly borne in mind on account of its bearing upon loss of head due to various causes. It is illustrated by the comparison of flow through a pipe of uniform diameter throughout its length and through a Venturi meter the ends of which are of similar diameter. While both may be discharging the same quantity of water, the flow in the former is uniform, in the latter, steady but non-uniform, due to its varying cross-section.

With few exceptions, the ordinary formulas relating to flow of water deal only with conditions of steady and uniform flow. Sizes, capacities and slopes are generally determined by assuming such conditions, yet they rarely exist in sewers and it is often necessary to give consideration to the effects of steady non-uniform flow, and also of unsteady flow. The former condition always exists when there is a change in velocity resulting from a change in slope or of cross-section or where there is a partial obstruction in the conduit, and its effects may be experienced for a considerable distance; the latter condition results from accretions to the quantity flowing, which are frequent in the case of most sewers.

In cases of unsteady or non-uniform flow, the only practicable method of computation is to assume the conduit divided into sections short enough so that the flow in each section may be assumed as steady and uniform without introducing serious error, and by successive approximations to obtain results close enough to meet the requirements of the problem. Some special cases of steady non-uniform flow frequently encountered, and sometimes of considerable importance, are discussed in a subsequent section of this chapter.

72. Equation of Continuity.—The discharge of any conduit is given by the expression $Q = Av$.

If the flow is steady in any given conduit there follows the equation of continuity.

$$A_1v_1 = A_2v_2, \text{ etc.}$$

The term "velocity," when employed without qualification, is used throughout this discussion to signify the mean or average velocity in the entire cross-section. It is clear that as frictional resistance exists between the water and the walls of the pipe or conduit, the velocity of flow at these walls must be less than that in the center of the stream.

When there are no losses other than those due to friction, mean velocity is dependent upon, first, the available head or fall, and second, the resistance to the flowing stream.

The variation in the computed mean velocity of partially filled sewers at varying depths of flow is shown in Figs. 39 to 46.

The resistance to flow varies with the length, wetted perimeter, and cross-section of the pipe, conduit, or channel; the rugosity, or roughness, of its interior surface; the temperature and hence viscosity of the fluid; the velocity, and the condition of flow, uniform, steady or variable. The resistance was shown by Dubuat to be independent of the water pressure, thus establishing the essential difference of the frictional resistance between a fluid and a solid and the frictional resistance between two solids—the latter of which is dependent upon the weight or pressure of one solid upon the other.

73. Formulas for Flow in Pipes and Channels.—The various formulas for flow of water in pipes and channels are essentially empirical, and all of them apply to steady uniform flow, and consider only the losses due to friction.

In general, sewers are designed with the expectation that they will be full only under maximum flow conditions. The ordinary condition of flow is, therefore, that of an open channel, in which there is a free water surface in contact with air. When full, they are usually under no material pressure, except in the case of force mains and inverted siphons.

74. The Chezy Formula.—The Chezy formula was intended to be applicable either to open channels or to pipes under pressure, and is perhaps as satisfactory for one case as for the other. The other formulas discussed herein were derived and originally

applied either for open channels, or for pressure pipes, but their use has since been extended, and they may all be used with a fair degree of satisfaction for open or closed channels such as are ordinarily considered in sewerage work. The Chezy formula is:

$$v = C\sqrt{RS}$$

The coefficient C was originally supposed to depend only upon the character of the walls of the conduit. It has since been found that the size and shape of the channel, the slope, and possibly other conditions, affect the value of C.

The formula has long been the one most familiar to engineers, and as substantially all of the later results of experiments have been applied to it, as well as to some other formulas, the limits of its applicability have been better established than have those of any other formula for the flow of water in pipes, conduits, canals, and rivers.

The determination of the coefficient C under different conditions has received much study from hydraulicians. Indeed, several of the most widely used formulas are, in effect, the Chezy formula with additional terms for determining more readily the value of C.

The Chezy formula for the case of circular pipes flowing full may also be written in another form, which is attributed to Weisbach (see Coxe's translation of WEISBACH's "Mechanics," p. 866).

$$h_f = f\frac{L}{D}\frac{v^2}{2g}$$

in which $f =$ the coefficient of friction, which decreases with increase in pipe diameter and slightly with velocity of flow.

75. The Kutter Formula.—Among those who have given study to the correct determination of the coefficient C to be used in the Chezy formula for the flow of water in pipes, conduits, and channels, were the Swiss engineers Ganguillet and Kutter, of Berne, who have devised an empirical formula based upon the results of experiments.

In its general form, the formula, which is known as Kutter's, is

$$v = \left\{ \frac{41.66 + \dfrac{1.811}{n} + \dfrac{0.00281}{S}}{1 + \left(41.66 + \dfrac{0.00281}{S}\right)\dfrac{n}{\sqrt{R}}} \right\} \sqrt{RS}$$

This formula was devised for open channels, such as canals and natural streams. Its authors did not suggest its use for closed pipes under pressure, but it has been found reasonably satisfactory for those. It was, therefore, inferred that it was equally applicable to conduits and sewers, which are open channels (that is, have a free water surface in contact with the air) except when absolutely full. However, the cross-sections of such conduits when more than about half full are radically different from those of ordinary open channels and experiments have shown that, except when the velocities are very low, the formula does not give correct values of v for all depths in circular pipes unless the value of n is varied; therefore n is not merely a coefficient of roughness, but is also affected by the shape of the section and perhaps by other conditions. Comparison of actual velocities with those computed by the formula using a constant value of n are given in Section 82.

76. Values of *n* in Kutter's Formula.[1]—For coefficients of roughness, n, with their reciprocals Ganguillet and Kutter suggested the values in Table 31.

TABLE 31.—VALUES OF n IN KUTTER FORMULA RECOMMENDED BY GANGUILLET AND KUTTER

	n	$\frac{1}{n}$
1. Channels lined with carefully planed boards or with smooth cement	0.010	100.00
2. Channels lined with common boards	0.012	83.33
3. Channels lined with ashlar or with neatly jointed brickwork	0.013	76.91
4. Channels in rubble masonry	0.017	58.82
5. Channels in earth, brooks, and rivers	0.025	40.00
6. Streams with detritus or aquatic plants	0.030	33.33

These values are not sufficient to enable a proper choice to be made, and are not altogether consistent with the results

[1] For computing values of n from observed data, the formula becomes:

$$n = \sqrt{R}\left(\sqrt{\frac{1.811}{BC\sqrt{R}} + G^2} - G\right)$$

where

$$B = 41.66 + \frac{.00281}{S}$$

$$C = \frac{v}{\sqrt{RS}} \text{ and } G = \frac{C - B}{2BC}$$

TABLE 32.—R. E. HORTON'S VALUES OF n; TO BE USED WITH KUTTER'S FORMULA

Surface	Best	Good	Fair	Bad
Uncoated cast-iron pipe	0.012	0.013	0.014	0.015
Coated cast-iron pipe	0.011	0.012[1]	0.013[1]	
Commercial wrought-iron pipe, black	0.012	0.013	0.014	0.015
Commercial wrought-iron pipe, galvanized	0.013	0.014	0.015	0.017
Smooth brass and glass pipe	0.009	0.010	0.011	0.013
Smooth lockbar and welded "OD" pipe	0.010	0.011[1]	0.013[1]	
Riveted and spiral steel pipe	0.013	0.015[1]	0.017[1]	
Vitrified sewer pipe	0.010 0.011	0.013[1]	0.015	0.017
Common clay drainage tile	0.011	0.012[1]	0.014[1]	0.017
Glazed brickwork	0.011	0.012	0.013[1]	0.015
Brick in cement mortar; brick sewers	0.012	0.013	0.015[1]	0.017
Neat cement surfaces	0.010	0.011	0.012	0.013
Cement-mortar surfaces	0.011	0.012	0.013[1]	0.015
Concrete pipe	0.012	0.013	0.015[1]	0.016
Wood-stave pipe	0.010	0.011	0.012	0.013
Plank flumes:				
Planed	0.010	0.012[1]	0.013	0.014
Unplaned	0.011	0.013[1]	0.014	0.015
With battens	0.012	0.015[1]	0.016	
Concrete-lined channels	0.012	0.014[1]	0.016[1]	0.018
Cement-rubble surface	0.017	0.020	0.025	0.030
Dry rubble surface	0.025	0.030	0.033	0.035
Dressed ashlar surface	0.013	0.014	0.015	0.017
Semicircular metal flumes, smooth	0.011	0.012	0.013	0.015
Semicircular metal flumes, corrugated	0.0225	0.025	0.0275	0.030
Canals and ditches:				
Earth, straight and uniform	0.017	0.020	0.0225[1]	0.025
Rock cuts, smooth and uniform	0.025	0.030	0.033[1]	0.035
Rock cuts, jagged and irregular	0.035	0.040	0.045	
Winding sluggish canals	0.0225	0.025[1]	0.0275	0.030
Dredged earth channels	0.025	0.0275[1]	0.030	0.033
Canals with rough stony beds, weeds on earth banks	0.025	0.030	0.035[1]	0.040
Earth bottom, rubble sides	0.028	0.030[1]	0.033[1]	0.035
Natural stream channels:				
1. Clean, straight bank, full stage, no rifts or deep pools	0.025	0.0275	0.030	0.033
2. Same as (1), but some weeds and stones	0.030	0.033	0.035	0.040
3. Winding, some pools and shoals, clean	0.033	0.035	0.040	0.045
4. Same as (3), lower stages, more ineffective slope and sections	0.040	0.045	0.050	0.055
5. Same as (3), some weeds and stones	0.035	0.040	0.045	0.050
6. Same as (4), stony sections	0.045	0.050	0.055	0.060
7. Sluggish river reaches, rather weedy or with very deep pools	0.050	0.060	0.070	0.080
8. Very weedy reaches	0.075	0.100	0.125	0.150

[1] Values commonly used in designing (according to Horton; the authors' recommendations will be found on p. 124).

of more recent experiments. The best modern list of values of n is that given by Robert E. Horton,[1] which is reproduced as Table 32, as printed in King's "Handbook of Hydraulics."

The following values of n for sewer pipes, conduits and channels are recommended for use under reasonably good operating conditions:

	n
For sewers 24 in. or less in diameter	0.015
For sewers over 24 in. in diameter of best work	0.012
For sewers over 24 in. in diameter under good ordinary conditions of work	0.013
For brick sewers lined with vitrified or reasonably smooth hard burned brick and laid with great care, with close joints	0.014
For brick sewers under ordinary conditions	0.015
For brick sewers, rough work	0.017 to 0.020

Although many engineers employ $n = 0.013$ for the smaller sewers, the value $n = 0.015$ is to be preferred, in view of the possibility of rough pipe and poor pipe-laying, as well as the presence of branches and manholes, all of which will increase the resistance. If $n = 0.013$ is assumed, great care must be taken in construction and in specifying and accepting materials, to make certain that the character of construction required is obtained and other causes of resistance to flow must be eliminated as far as possible.

77. Limitations of Kutter's Formula.—Being essentially an empirical formula, based upon actual gagings, it is of importance to remember the limits within which observations have been made, and further to remember that while velocity varies approximately as the square root of the head under velocities corresponding to the ordinary conditions of flow, it varies more nearly directly as the head when velocity is very low. Within the ordinary velocity limits of from 1 to 6 ft. per second, the formula is most trustworthy. It is fairly reliable up to 10 ft. per second velocity. For special cases which may be outside of the range of the formula, such as 20 ft. per second or higher velocity, the engineer should refer to the original data, published in Hering and Trautwine's translation of Ganguillet and Kutter's work, and those of other writers upon hydraulics since that time. See also Sections 75 and 82.

[1] *Eng. News*, 1916; **75**, 373.

Hughes and Safford[1] have summed up the application of this formula in an excellent manner as follows:

> There is a wide range in the magnitude of the streams on which this formula is based (from hydraulic radii of 0.28 to 74.4 ft.); but a study of the data on which the formula is based, as given in the authors' book, has led to the following conclusions:
>
> That, for hydraulic radii greater than 10 ft., or velocities higher than 10 ft. per second, or slopes flatter than 1 in 10,000, the formula should be used with great caution. For hydraulic radii greater than 20 ft., or velocities higher than 20 ft. per second, but little confidence can be placed in results.
>
> That, considering the variable accuracy of the data on which the formula is based, results should not be expected to be consistently accurate within less than about 5 per cent.
>
> That, for any slope steeper than 0.001 the values of C computed for $S = 0.001$ may be used with errors less than the probable error in the ordinary use of Kutter's formula.
>
> That, between slopes of 0.001 and 0.0004 the maximum variation (at the extreme values of n and R) in C is about 4 per cent; for such values as fall within the range of ordinary practice, the maximum variation is but 2 per cent.
>
> That between slopes of 0.0004 and 0.0002 the maximum variation is about 5 per cent, but for such values as fall within the range of ordinary practice the maximum is less than 3 per cent.

78. Simplified Kutter Formula.—Prof. H. E. Babbitt[2] has called attention to the fact that the omission of the term $0.00281/S$ in the Kutter formula will generally cause no greater differences in results, as compared with the complete formula, than those accompanying a difference of 0.001 in the value of n. This modification is, therefore, justified in some cases, and the work of computation may be greatly reduced, as compared with that required for the original formula.

79. The Manning Formula.—Robert Manning[3] published a new and simpler formula for flow in open channels, in which the value of C in the Chezy formula is

$$C = \frac{1.486}{n} R^{1/6}$$

This form was adopted in order to use the same values of n as in Kutter's formula.

[1] "Hydraulics," 1st. Ed., 343.
[2] *Eng. Contr.*, 1922; **57**, 128.
[3] *Trans.*, Inst. Consulting Eng. Ireland, 1890.

The Manning formula may also be written

$$v = KR^{2/3}S^{1/2}$$

in which

$$K = \frac{1.486}{n}$$

An extensive comparison of values of n, computed by the Kutter and Manning formulas from the results of experiments, was made by King,[1] from which he concludes that . . . "The agreement . . . between Manning's n and Kutter's n is extremely close, and . . . the two formulas (using the same value of n in each case) give results agreeing well within the limits of uncertainty which must exist in the selection of n, under all ordinary conditions."

King also gives a table, containing the computed values of Kutter's n and Manning's n for identical values of C in the Chezy formula which shows how closely they agree within the range of ordinary experience. It should be noted, however, that the differences are most marked when the values of R are small, and consequently the divergence between the results of the Manning and Kutter formulas would be more marked in the case of small than of large sewers, if identical values of n were used.

80. The Hazen and Williams Formula.—Of late years, several exponential formulas for the flow of water in pipes and conduits have been developed. Of these the most important is that developed in 1902 by Allen Hazen and Gardner S. Williams, which agrees closely with observed results and has the great merit that it can be applied with facility through the special slide rule designed and graduated for the solution of problems by it. Tables have also been prepared covering its application.[2] Inasmuch as careful comparison of this formula has been made with the Kutter formula, and as the use of the slide rule is not only convenient but effects a very considerable saving in time in making many hydraulic computations, this formula is of particular importance. While this formula has had application most often to pipes discharging under pressure, it may also be used in sewer computations.

[1] "Handbook of Hydraulics," 2nd. Ed., p. 262.

[2] WILLIAMS and HAZEN: "Hydraulic Tables," 3rd. Ed., 1920.

The Hazen and Williams formula is

$$v = CR^{0.63}S^{0.54}\,0.001^{-0.04}$$

The authors of the formula say of it,[1]

The exponents in the formula used were selected as representing as nearly as possible average conditions, as deduced from the best available records of experiments upon the flow of water in such pipes and channels as most frequently occur in waterworks practice. The last term, $0.001^{-0.04}$, is a constant, and is introduced simply to equalize the value of C with the value in the Chezy formula, and other exponential formulas which may be used, at a slope of 0.001 instead of at a slope of 1.

This formula may also be written, (since $0.001^{-0.04} = 1.318$),

$$v = 1.318CR^{0.63}S^{0.54}$$

With regard to the coefficients to be used in this formula in general design, Williams and Hazen suggest the following values for C:

140 for new cast-iron pipe when very straight and smooth

130 for new cast-iron pipe under ordinary conditions

100 for old cast-iron pipe under ordinary conditions, this value to be used for ordinary computations anticipating future condition

110 for new riveted steel pipe

95 for steel pipe under future conditions

140 for new lead, brass, tin, or glass pipe with very smooth surface

130 to 120 ditto, when old

120 for smooth wooden pipe or wooden stave pipe

140 for the masonry conduits of concrete or plaster with very smooth surfaces and when clean

130 ditto, after a moderate time when slime-covered

120 ditto, under ordinary conditions

110 for cement-lined pipe (Metcalf)

100 for brick sewers in good condition

110 for vitrified pipe sewers in good condition.

The relation between the value of C in the Williams and Hazen formula and the C of the Chezy formula may be found by equating the value of S in these two formulas, which gives the equation

$$C(\text{Chezy}) = 1.1506C^{0.9259}v^{0.0741}D^{0.0833}$$

[1] WILLIAMS and HAZEN: "Hydraulic Tables," pp. 1 and 2.

For ordinary work the Kutter and the Hazen-Williams formulas agree closely enough to permit the use of either. That is, the difference in sizes of sewers based on the two formulas is usually within the range of commercial pipe sizes, as shown in Table 33. For especially large and important work, special studies are warranted and less reliance should be placed upon coefficients and the arbitrary selection of sizes from the diagrams ordinarily used.

TABLE 33.—COMPARISON OF SEWER SIZES IN INCHES RESULTING FROM THE USE OF THE KUTTER AND HAZEN-WILLIAMS FORMULAS

Quantity of sewage		Slope of sewer per 1,000					
		0.1		1.0		10.0	
Cubic feet per second	Million gallons daily	Kutter	Hazen and Williams	Kutter	Hazen and Williams	Kutter	Hazen and Williams
2	1.29	24	24	15	15	10	10
10	6.46	42	42	24	27	18	18
50	32.30	77	78	50	49	33	30
2	1.29	25*	25*	16*	16*	10	10
10	6.46	45	45	30	30	20	18
50	32.30	81	84	55	52	33	33

Note: In the computations for the first three lines of figures, n was taken as 0.013 and c as 120; for the last three lines, n was taken as 0.015 and c as 100.

* These are not commercial sizes of sewer pipes, and sewers of these sizes would not be used. Ordinarily the 24- and 15-in. sizes computed with $n = .013$ would be used, and the sewer allowed to run under a slight head; if this were not allowable, 27- and 18-in. sizes would have to be used.

When n is taken as 0.013 in Kutter's formula, C in the Hazen-Williams formula may be taken as 120; when n is taken as 0.015, C may be taken as 100.

Kutter's formula is generally used with the aid of diagrams giving not only the velocity but also the discharge. Figs. 32, 33 and 34 are examples of such diagrams for circular sewers, Fig. 35 for a semi-elliptical sewer, and Fig. 36 for an egg-shaped sewer.

Fig. 37, prepared by Fred C. Scobey,[1] is for use in solving problems of a more general nature involving Kutter's formula.

[1] For permission to reproduce this diagram the authors are indebted to the U. S. Department of Agriculture and Mr. Scobey. It was first printed in its present form in CREAGER and JUSTIN's "Hydroelectric Handbook," 1927.

The discharge is equal to the product of the mean velocity by the area of the cross-section of the flowing stream.

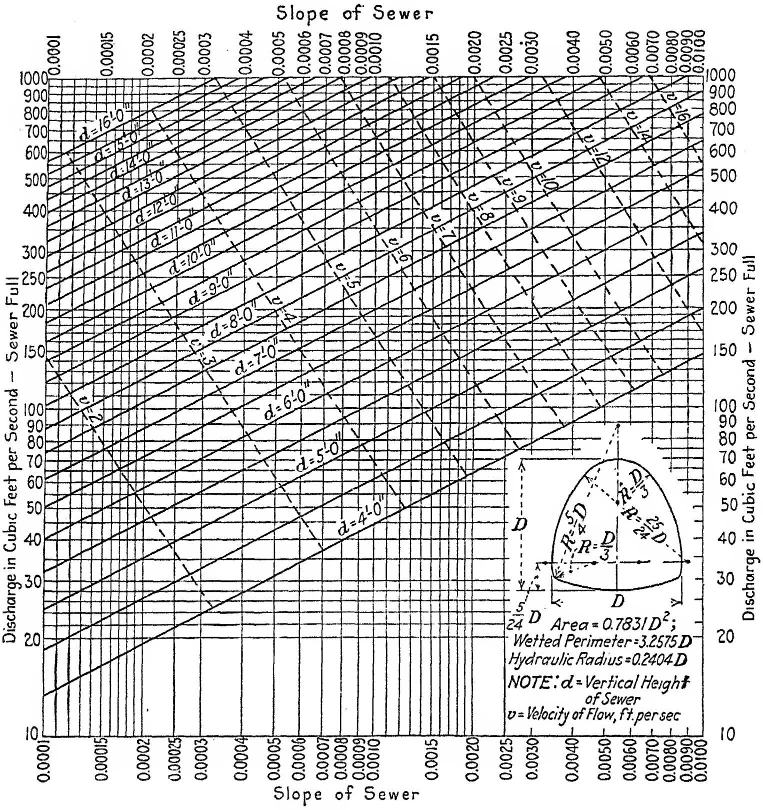


FIG. 35.—Capacity of semi-elliptical sewers, M. & E. standard section. $n = .013$.

81. Flow in Partly Filled Sections.—In many of the problems arising in sewer design it is necessary to know the velocity and discharge when a sewer is partly filled. The relations between hydraulic elements for flow at full depth and at other depths, computed according to the Kutter formula, may be found from diagrams like those of Figs. 38 to 42 inclusive.[1] Such a diagram is usually prepared by plotting the cross-section carefully and

[1] These diagrams show, in each case, the outline of one-half of the sewer section and for each proportionate depth the corresponding proportionate values of the area, mean velocity, and discharge, according to the formula; and two of the diagrams also show the hydraulic radius. It should be noted that the names by which these sections are known are descriptive of the shape of the portion above the invert. Thus the so-called "rectangular" section consists of a V-shaped or curved invert surmounted by a rectangle; the semi-elliptic and semi-circular sections have curved inverts surmounted by semi-elliptic or semi-circular arches, etc.

then measuring with a planimeter the area of the stream when the section is filled to various depths. These areas divided by the wetted perimeters, also carefully measured, give the mean hydraulic radii. A slope and a coefficient of roughness, n, are then assumed and, using the Kutter formula, the velocities and discharges for the different depths are computed and plotted in terms of the velocity and discharge when flowing full. The curves thus obtained show the computed relative velocities and discharges at different depths as compared with those of the full section. These diagrams, although mathematically correct only for the data given, are sufficiently close for other sizes, slopes and friction factors to be of general use, and as a rule the difference may be neglected.

Attention is particularly called to the egg-shaped section, Fig. 39. This was introduced about 1846 by John Phillips in order to obtain adequate velocities in combined sewers when the depth of sewage in them was small. When the quantity of sewage is not more than two-tenths of the total capacity of the sewer, the velocity in an egg-shaped sewer will be somewhat greater than in a circular sewer of the same total capacity. The depth of the sewage will also be greater and there is a resulting better flotation for solids. These differences are not of great magnitude, however.

82. Discharge and Velocity at Different Depths.—The capacity and velocity in a 12-in. sewer when full are 2 cu. ft. per second and 2.5 ft. per second respectively, the slope being 0.005 and n being 0.015. If n is constant, when the depth of flow is 0.1 ft., the discharge will be but 0.06 cu. ft. per second or 3 per cent of its capacity when full. At that depth, however, the velocity of flow will be 33 per cent of that of the full sewer, or 0.84 ft. per second. At a depth of 0.2 ft., the discharge will still be small, but the velocity will have increased substantially, to 1.38 ft., 55 per cent of the velocity when the sewer is full. In other words, the velocity increases much more rapidly with increasing depth than does the discharge.

That the Kutter formula with a constant value of n is not applicable throughout the range of depths, at least in the case of circular conduits of small and moderate size, (except when velocities are very low) has been noted in Sec. 75.[1] Diagrams of hydraulic elements such as those in Figs. 38 to 42 are therefore

[1] See also *Eng. News-Rec.*, 1929; **103**, 253.

only approximately correct, and should be used only with an understanding of their limitations. Experimental determina-

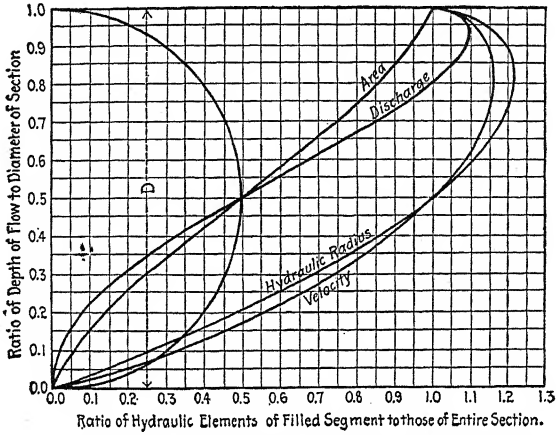


FIG. 38.—Hydraulic elements of circular section by Kutter's formula. $n = 0.015$; $s = 0.005$; $D = 1$ ft. Area $= 0.785D^2$; Wetted Perimeter $= 3.1416D$; Hydraulic Radius $= 0.250D$.

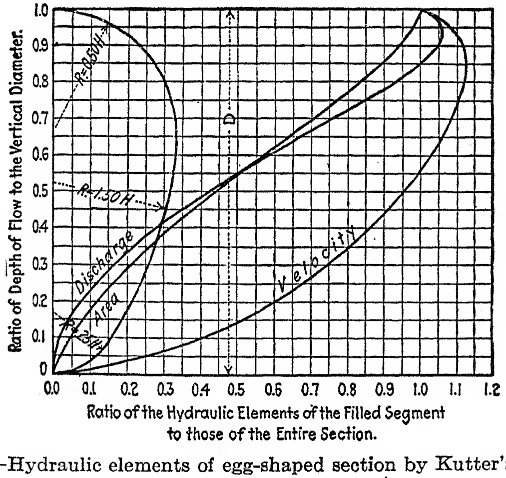


FIG. 39.—Hydraulic elements of egg-shaped section by Kutter's formula. $n = 0.015$; $s = 1/1{,}600$; $H = 4$ ft.; $D = 6$ ft. $= 1.254$ diam. equiv. circle; $H = 0.836$ diam. equiv. circle; $A = 1.1485\ H^2 = 0.5105D^2$; $R = 0.2897H = 0.1931D$.

tions[1] of velocities at various depths in circular pipes from 4 to 42 in. in diameter, have given results generally similar to those

[1] YARNELL and WOODWARD: *Bull.* 854. U. S. Dept. of Agriculture; 1920.

WILCOX, E. R.: *Bull.* 27, Engineering Experiment Station, University of Washington; 1924.

SCOBEY, F. C.: *Bull.* 852, U. S. Dept. of Agriculture; 1920.

shown on Fig. 43, which shows the average velocities actually observed in an 8-in. sewer pipe at various slopes. There is also

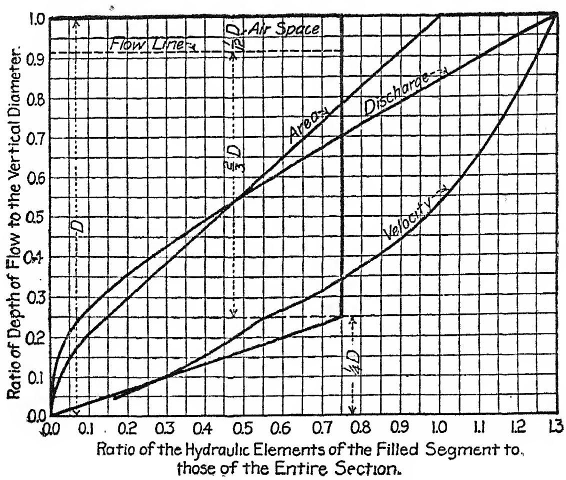


FIG. 40.—Hydraulic elements of rectangular section by Kutter's formula. $n = 0.013$; $s = 0.001$; $D = 6$ ft.

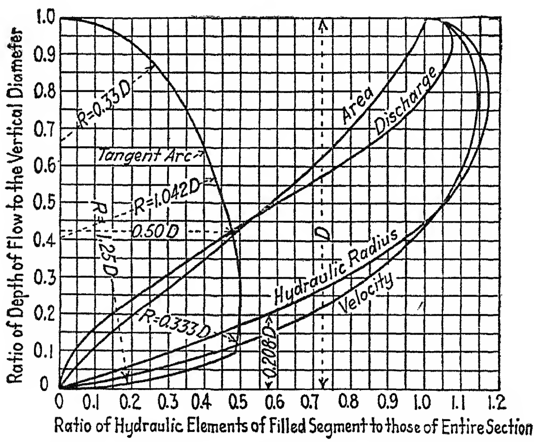


FIG. 41.—Hydraulic elements of Louisville[1] semi-elliptic section by Kutter's formula.

$n = 0.013$; $s = 0.0003$; $D = 7\frac{1}{2}$ ft.; Area $= 0.785D^2$; Wetted perimeter $= 3.26D$; Hydraulic radius $= 0.242D$.

shown for comparison a computed curve of average velocities for the steepest slope. It will be seen that the maximum average

[1] The internal dimensions of the Louisville and Metcalf and Eddy semi-elliptic sections are identical.

velocities were obtained in some cases when the pipe was entirely filled.

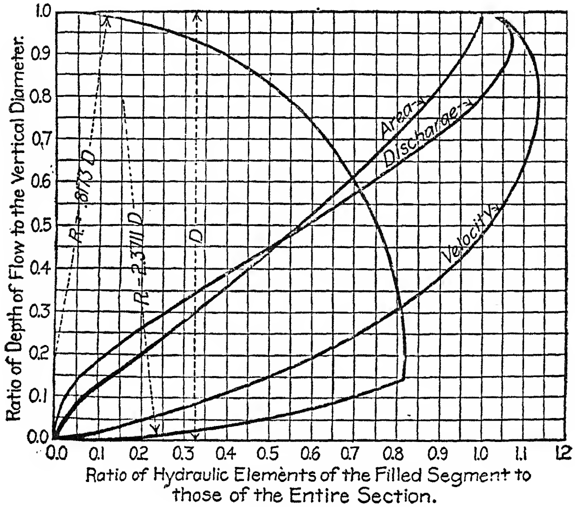


FIG. 42.—Hydraulic elements of semi-circular section by Kutter's formula. $n = 0.013$; $s = 0.001$; $D = 9$ ft. 2¼ in., Area $1.2697D^2$; Hydraulic radius $= 0.2946D$.

Most experimental determinations of the values of n for sewers have been made in conduits but partly filled and with

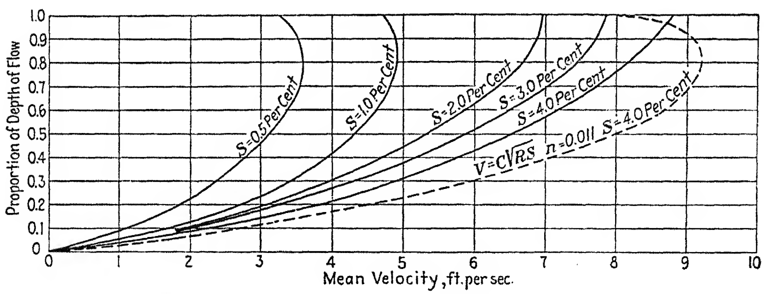


FIG. 43.—Relation between mean velocity and depth of flow in 8″ vitrified clay sewer pipe as determined by quantity-area method. (*Bull.* 27, *Univ. of Washington Engineering Experiment Station.*)

slopes which are common in sewer work, and the values recommended in Sec. 76 are believed to be reasonable for conditions

corresponding to depths of less than $0.6D$ and for such slopes as are usually obtained. The experiments referred to above indicate that, at least in some cases, smaller values of n will obtain for greater depths of flow, and therefore the corresponding capacities will be greater than those computed. Therefore the error, if any, is on the side of safety, and the continued use of such diagrams as those in Figs. 38 to 42 is justified in design, but not in attempting to estimate the quantity flowing from observed depths and slopes.

VELOCITIES AND GRADES

83. Distribution of Velocity in Cross-section.—The moving body of water in any conduit travels at an average velocity v which is the mean of the velocities of all the filaments in a cross-section. These velocities are not uniform, but are least for the filaments in contact with the enclosing walls of the conduit. In a straight, closed conduit or pipe having a perimeter of uniform character, where the frictional effect of the walls is everywhere the same, the maximum velocity will be found at the center of the cross-section, or in the axis of the conduit. In an open conduit, the maximum velocity will be found at the greatest possible distance from all surfaces against which there is friction; the greater the friction upon any surface, the greater will be the distance of the point of maximum velocity from that surface. The water surface, or plane of contact between the flowing water and the atmosphere, is not a frictionless surface (except when the air is moving in the same direction and at the same velocity) and, consequently, the maximum velocity is below the surface, but at a much less distance than from the walls of the enclosing channel.

An examination of the results of a large number of determinations of variation of velocity in the cross-section of pipes and open channels has shown that, in general, the curve of variation of velocity in any axial section of a closed pipe, or vertical section in an open channel, is approximately parabolic in form, with the axis of the parabola approximately in the thread of maximum velocity. The filaments in contact with the surface of the conduit move at velocities considerably less than the mean velocity in the cross-section.

84. Ratio of Mean to Maximum Velocity.—In straight closed pipes, the mean velocity has been found to be about 0.85 of the

maximum velocity in the cross-section (the center velocity) and to be located about three-fourths of the radius from the center. The velocity at the perimeter is about one-half the center velocity.

In open channels, the mean velocity *in any vertical section* is usually between 0.80 and 0.95 of the surface velocity in that vertical. The thread of mean velocity is usually between 0.55 and 0.65 of the depth below the surface, and the average of the velocities at 0.2 and 0.8 depth usually gives the mean velocity in the vertical within about 2 per cent.

The ratio of the mean velocity in the cross-section to the maximum surface velocity is not a constant, but it has usually been found to lie between 0.70 and 0.85 in all types of open channels, and the value 0.80 has often been used as a rough approximation.

85. Velocity of Flow.—The velocity of flow depends upon the slope and the resistance to flow. The slope in sewer design always refers to the inclination of the hydraulic grade line, as heretofore defined, often called the hydraulic gradient. In a large proportion of the sewers which the engineer is called upon to design, the distinction between the slope of the invert and the hydraulic gradient is unimportant, and it is generally disregarded in designing separate sewers and in estimating the dry-weather discharge of combined sewers. But it must be considered in estimating the discharge of combined sewers while carrying storm water, and in all cases it should be kept in mind that, while for construction purposes the slope of the sewer is that of the invert, it is the hydraulic gradient when questions of velocity and discharge must be solved. Serious difficulty in the operation of the sewer systems of some cities has resulted from the failure to use the hydraulic gradient in computing the capacities of the larger sewers, particularly those affected during portions of the day by the entrance of tidal water into their lower ends.

If a uniform velocity is to be maintained in a long sewer which receives the flow from branches at intervals and the cross-section of which must therefore be increased at each branch, the slope will be flatter as the size increases, and the profile will approximate that of a concave curve. If the sewer is to be full but not under pressure, the hydraulic grade line must coincide with the crown of the sewer, and there will be a drop in the invert at each change in size. This condition is illustrated in Fig. 44.

If no drop in the invert were provided, and the hydraulic grade line remained in the same position (as it would if the same sizes of sewer were used) then the smaller sewers would all be under pressure, as shown in Fig. 45. The total amount of fall between the invert elevations at the upper and lower ends is greater in Fig. 44 than in Fig. 45; but the fall between the elevations of the hydraulic gradients at the two ends is the same in either case.

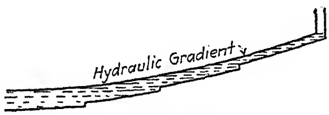


Fig. 44.

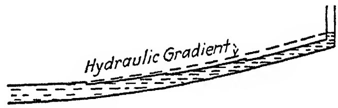


Fig. 45.

86. Velocity and Transporting Power of Water.—The transporting capacity of water, due to its velocity, plays an important part in the disposal of sewage by dilution and diffusion and in preventing clogging of sewers and local formation of sludge banks, as a result of the settling out of the heavier particles of sewage. It has been shown[1] that the weight of objects which can be transported by water varies as the sixth power of its velocity, so that if the velocity be doubled the transporting capacity is 64 times as great. Therefore, any influence which tends to check the velocity at any point immediately results in substantial reduction of its transporting power, and the subsequent deposition of particles which had been carried along readily by the current of greater velocity. Transporting capacity here applies to the size and weight of particle moved by the water rather than the total amount of material carried.

Many of the published statements relating to the transporting capacity of streams are almost valueless, owing to the lack of exact knowledge of the velocity near the bottom of the stream,[2] which, together with the character of the material composing the bottom and the depth and, hence, the pressure of water upon it, are the most important elements in the problem of erosive action. Comparisons with average velocities are of slight significance. Freeman has called pointed attention to these facts in his Charles River Dam report, and has cited the opinion of Hiram F. Mills

[1] Merriman: "Hydraulics," 9th Ed., 340.

[2] In general, the velocity near the bottom of an open channel is between 40 and 80 per cent of the mean velocity, depending largely upon the roughness of the bottom; two-thirds of the mean velocity may be taken for a rough approximation to the bottom velocity in sewers.

in regard to the misuse of the observations of Dubuat, who made experiments in 1780 upon the capacity of a stream in a wooden trough to move particles to its bottom. All of these observations failed to take into account the varying velocities of flow in vertical section. Many engineers who have made use of the results of those experiments have failed to recognize this fact, as well as the effect of the character of the material, the coating of slime or colloidal surface which forms upon the bottom, and the effect of the pressure of the water upon the bottom material.

The most extended study of the transportation of debris by flowing water made in this country is that of Grover K. Gilbert, described in Professional Papers 86 and 105 of the U. S. Geological Survey. Extensive studies have also been carried out in the hydraulic laboratories of Germany, but most of them have had to do with the amount rather than the size of particles carried under various conditions of flow and of stream channel.

Experiments have shown that the effect of the specific gravity on susceptibility to the velocity of water is as recorded in Table 34. With a material of definite uniform specific gravity, the size and shape of the particle determines the velocity which will permit it to settle or which will dislodge it and carry it along. The Metropolitan Sewerage Commission of New York assumed in its report of 1910 that the velocities given in Table 35 were necessary to move solid particles.

TABLE 34.—EFFECTS OF SPECIFIC GRAVITY ON TRANSPORTATION BY VELOCITY OF WATER

Nature of bodies	Specific gravity	Velocity in feet per second required to transport
Coal	1.26	1.25 to 1.50
Coal	1.33	1.50 to 1.75
Brickbat	2.00	1.75 to 2.00
Piece of chalk	2.05	
Oolite stone	2.17	
Brickbat	2.12	2.00 to 2.25
Piece of granite	2.66	
Brickbat	2.18	2.25 to 2.50
Piece of chalk	2.17	
Piece of flint	2.66	2.50 to 2.75
Piece of limestone	3.00	

In general, it is found that a mean velocity of 1 ft. per second, or thereabouts, is sufficient to prevent serious deposition of sewage solids upon tidal flats, if the solids are reasonably comminuted. It is not enough, however, to prevent deposition of mineral matter such as sand and gravel.

TABLE 35.—VELOCITIES NECESSARY TO MOVE SOLIDS
(Metropolitan Sewerage Commission, New York)

Kind of material	Velocity required to move on bottom	
	Feet per second	Miles per hour
Fine clay and silt	0.25	about $\frac{1}{6}$
Fine sand	0.50	about $\frac{1}{3}$
Pebbles half inch in diameter	1.0	about $\frac{2}{3}$
Pebbles 1 in. in diameter	2.0	about $1\frac{1}{3}$

87. Minimum Velocities.—The sewage should flow at all times with sufficient velocity to prevent the permanent settlement of solid matter in the sewer. As already stated, the transporting capacity of flowing water varies as the sixth power of the velocity, and, therefore, there is a certain velocity which is just sufficient to carry along certain classes of suspended matter, and if this be checked very little, these substances will be dropped. It is manifestly unwise to approach too closely to this velocity, for a slight increase in the roughness of the interior of the sewer or the presence at any point of deposits of heavy substances might cause enough decrease in the velocity to produce the undesirable settling of the suspended matter. Moreover it is important to note that the form and adhesive nature of some of the suspended matter are such that if this matter is once deposited it has a tendency to remain on the invert, even when the velocity of flow increases again to a rate ample to keep the material in suspension had it not been deposited.

While it is the velocity near the bottom of the sewer which is significant in studying the transporting power of flowing water, it is generally agreed that a mean velocity of $2\frac{1}{2}$ ft. per second will ordinarily prevent deposits in combined sewers and 2 ft. per second will generally prevent their occurrence in separate sewers, under favorable conditions. These are minimum figures. It is desirable to have a velocity of 3 ft. per second or more wherever practicable. At least 3 ft. velocity should be obtained in inverted siphons, access to which, for cleaning.

is always difficult. Slopes giving mean velocities as low as 1½ ft. per second have been used successfully in some special cases in sewers, but the latter must be built and their interior surface finished with great care in order to achieve successful working conditions. Repeated removal of sludge and hard materials from sewers is expensive, and if such deposits are not cleaned out they may cause troublesome conditions, gradually increasing in their annoying character; it is desirable, therefore, to use grades which will give self-cleansing velocities in all cases, even in those where the resulting increase in cost of construction due to steeper slopes will involve fixed charges greater than the added cost of maintaining the sewers if laid on flatter slopes, for if such maintenance work be neglected, a substantial deposit may form, and then the sewer cannot perform its functions properly and in emergency may fail to carry the sewage and storm water tributary to it, resulting in damage to property.

88. Examination of Sewer Design with Reference to Minimum-flow Conditions.—It is necessary, after designing a sewer, particularly a trunk or intercepting sewer, for a given service in the future, to consider the actual conditions of operation likely to arise at times of minimum flow during the first few years after its construction, in order to make certain that the velocities will not be so low, for significant periods of time, as to cause serious deposits in the sewer, the removal of which would involve unwarranted cost. The construction of a sewer to serve for a long period would be unwarranted if this cost of cleaning should exceed the cost of building a smaller sewer in the first instance, to serve for a shorter period of time and until the anticipated growth had developed in some degree, and of then building a second sewer to take care of the additional sewage flow resulting from the added growth. While the latter plan would involve greater cost of construction, enough might be saved in fixed charges and in the cost of operation, in the early years of the use of the sewer, to more than offset this increased cost.

When slopes are comparatively flat, it is desirable that the sewer sections and slopes be so designed that the velocity of flow will increase progressively, or at least be maintained uniform, in passing from the inlets to the outlet of the sewer, so that solids washed into the sewer and transported by the flowing stream may be carried through and out of the sewer, and not be

dropped at some point owing to a decrease in velocity. It is, however, seldom possible to fully attain this condition, due to topographical characteristics. In general, the minimum slopes givin in Table 36 for small pipe sewers in the seperate system have been found safe, though steeper grades are always desirable.

TABLE 36.—SLOPES REQUIRED TO PRODUCE 2 FT. PER SECOND VELOCITY IN PIPE SEWERS FLOWING FULL

(Based upon Kutter's Formula with $n = .013$)

Diameter, in.	Fall in ft. per 100 ft.
4	1.2
6	0.6
8	0.4
10	0.29
12	0.22
15	0.15
18	0.12
20	0.10
24	0.08

89. Maximum Velocities.—The erosive action of suspended matter depends not only on the velocity with which it is carried along the invert of a sewer but also on its nature. As it is this erosive action which is the most important factor in determining the safe maximum velocities of sewage, the character of the suspended matter must be considered when designing combined sewers where velocities will probably be high. For instance, velocities which caused serious erosion of the inverts of brick sewers at Worcester, Mass., caused no appreciable wear at Louisville, Ky. The difference is due to the fact that the street detritus entering the combined sewers at Worcester is made up largely of particles of hard quartz, while that at Louisville is largely clay and particles of limestone, having little effect upon rather soft brick, much less resistant than that used in the Worcester sewers.

Vitrified clay pipe resists abrasion well, and when it is used there is rarely any occasion to set a maximum velocity which must not be exceeded by the sewage passing through it. Sewer brick differs so greatly in quality that it is not possible to lay down any general rule for permissible maximum velocities where it is used. It is not advisable to use it for inverts where the mean velocity is over 8 ft. per second and where the street grit washed into the sewers contains much silica. If the street grit is soft

like that at Louisville or St. Louis, there will probably be little erosion of sewer brick with sewage flowing at velocities of 10 or 12 ft. per second. Where concrete sewers are constructed it is desirable to have the inverts given a hard finish, preferably a granolithic surfacing if velocities exceed 5 ft., and if the velocities exceed 8 ft. it is prudent to line the sewers with vitrified paving brick. Vitrified tile liner plates are being used to some extent in concrete sewers, both for protection against erosion and against attack by chemicals in the sewage.

Maximum velocities need rarely be considered except in combined sewers or storm-water drains where the grades are such that the velocity of flow may become excessive. The maximum velocity should always be ascertained for such sewers in order to be certain that the designer of the cross-sectional details knows whether he must provide against erosion. The greatest objection to high velocities in small pipe sewers is that with reduction in the volume of sewage flowing and consequent decrease in its depth, there are likely to be left stranded on the inverts, where they may become so firmly lodged that the next rush of sewage will not detach them, large floating substances which at times enter all separate sewerage systems. Theoretically rags, old brushes, pieces of wood, corncobs and such things should not be allowed to enter separate sewers, but they are sometimes thrown into the house fixtures and may be left on the invert of a small pipe sewer in which the sewage flows intermittently in swift flushes, as is likely to be the case near the upper ends where the grades are steep. It is for this reason that in some European cities are maximum grades of 1:15 and 1:20 are prescribed for 6-in. house connections and small lateral sewers respectively.

LOSSES OF HEAD FROM CAUSES OTHER THAN FRICTION

90. Velocity Head.—Strictly speaking, the head required to produce velocity, $v^2/2g$, is not lost, as it merely corresponds to the transformation of potential energy into kinetic energy, and theoretically it should be recoverable when the velocity is checked. This is the case, to a considerable extent, with closed pipes under pressure, but, in open-channel conditions, it is rarely possible to recover a material part of the velocity head, except in the "hydraulic jump." It is usually advisable in sewerage work to consider the head used in producing velocity as lost head, and it is necessary to provide, in every case, sufficient head to develop

velocity, in addition to that required to overcome friction and other resistances. At velocities normally found in sewers in flat territory, the velocity heads may be inconsiderable, but this hydraulic element should be considered in design, and rejected only if relatively unimportant in the problem under analysis, as failure to provide for it has sometimes led to serious loss of capacity, unexpected choking and overflow of sewers, and other inconvenience.

Some cases which require special attention in this particular may be cited. Where the flow at low velocity in a large sewer is to discharge into a smaller sized sewer in which the velocity will be high; in grit chambers, where sewage moving slowly through the enlarged chambers must increase its velocity to flow away through an outlet passage or conduit of smaller dimensions; in effluent conduits or channels leading away from sedimentation tanks; at restrictions in conduits, such as gates or contractions necessary to pass other structures; in short, any cases where velocity must be built up to initiate flow in a channel or conduit. These are the most important head losses encountered in the design of the hydraulic features of disposal plants and require careful consideration.

Recovery of Velocity Head.—When velocity is reduced by enlargement of a channel or conduit a portion of the change

Fig. 37.—Recovery of Velocity Head in Transition Sections, from Experiments by U. S. Reclamation Service

Location and name	Velocity, feet per second		Percentage of velocity head recovered	Discharge, cubic feet per second
	Up-stream end	Down-stream end		
North Platte Project	5.79	1.64	54	150
	5.87	1.70	25	105
Hope Creek Flume	5.67	1.65	28	99
Uncompahgre Valley Project, Happy Canyon Flume[1]	6.47	2.85	100 nearly	361
Tieton Main Canal Tunnel Outlets:				
1. Trail Creek	12.12	8.06	84	271
2. Columnar	11.42	7.98	60	261
3. Tieton	11.58	7.60	68	286
4. North Fork	10.25	2.62	100	286
Okanogan Project:				
Experiment Number IV	5.39	2.61	50	51
Experiment Number VI	3.31	2.46	62	

[1] Transition unusually favorable for recovery.

in kinetic energy may sometimes be recovered as potential energy, manifested by a rise in water surface, provided the change in section is smooth and gradual. The U. S. Reclamation Service has observed the extent of this recovery in numerous instances, (see Table 37), where irrigation canals have been contracted in section and then enlarged again further downstream.

The principle of this recovery of velocity head is well illustrated in an analogous problem, that of water flowing under pressure in the Venturi meter. In it the outlet cone is gradually expanded so as to recover a large part of the difference in velocity head between throat and outlet.

Entry Head.—In the case of a pipe leading from the side of a tank, the entry head loss has been found to be $0.505\frac{v^2}{2g}$, and is in addition to the velocity head. This is for the case where the corners are square and sharp. By suppressing or reducing the contraction resulting from the orifice, as by rounding the corners, the entry loss may be materially reduced, perhaps to $0.1\frac{v^2}{2g}$.

In the case of an open channel or conduit leading from a reservoir, the conditions are similar, but experimental data are lacking. It is probably safe to estimate the loss on the same basis as in a closed pipe.

Sudden Reduction of Cross-section.—In addition to the difference in velocity heads $\frac{v_2^2 - v_1^2}{2g}$, there is a further loss due to the resistance resulting from the disturbed flow conditions. Experiments on pipes have shown this additional loss to range from almost nothing to approximately $0.5\frac{v^2}{2g}$, (v being the velocity in the smaller pipe), depending on the ratio between the diameters, the coefficient being larger as the difference in diameter increases. There are no experimental data for open channels, but it seems reasonable to assume that similar relations exist.

Sudden Enlargement of Cross-section.—Archer has shown[1] that experimental data on losses due to sudden enlargements of closed pipes may be expressed by the formula

$$\text{loss} = 1.098\frac{v_1^{1.919}}{2g}\left(1 - \frac{A_1}{A_2}\right)^{1.919}$$

[1] *Trans.*, Am. Soc. C. E., 1913; **76**, 999.

King and Wisler's "Hydraulics" contains a table (p. 161) showing that this is equivalent to amounts ranging from nearly nothing to the velocity head in the smaller pipe ($v_1^2/2g$).

In the case of an open channel, very little of the velocity head can be recovered.

Other disturbances to flow are caused by bends and partly closed valves in pipes; and by changes in direction, piers, side inlets, bulkheads, and the effect of wind upon the free surface of the liquid in open channels.

Valves.—Many experiments upon the head loss resulting from partly closed valves in pipes indicate that in general the loss ranges from $0.2v^2/2g$ to $13.5v^2/2g$ (where v is the velocity in the pipe), as the ratio of the area of opening to that of the pipe decreases from 0.9 to 0.1, being about $2.7v^2/2g$ when the area of the opening is half that of the pipe.

Curves.—Many experiments upon loss of head due to curves in pipes have been made. Most of them, however, have been upon changes in direction amounting to 90 deg. Experiments in which the deflection differed from 90 deg. are too few to warrant definite conclusions, except for conditions similar to those of the experiments. In general, it appears probable that even a slight change in direction produces a condition of disturbed flow which increases the frictional resistance, and that the length in which such disturbed conditions exist is of greater significance than the sharpness of the deflection; in other words, a curve of short radius and correspondingly short length of curve is likely to result in smaller loss of head resulting from curvature than a curve of long radius with the same change of direction. There are limits, however, beyond which the opposite effect is probable.

The same conclusions relative to the effect of curves in open channels seem to be justified. H. P. Eddy has shown[1] that none of the formulas which have been proposed for loss due to curvature is applicable to open-channel conditions, and that such fragmentary data as are available for those conditions indicate that the effect of curvature has generally been equivalent to an increase in the value of n by an amount varying from 0.003 to 0.005 in the sections containing much curvature. In the present state of our knowledge, it seems logical to allow in design for the effect of curvature by a change in the coefficient of roughness.

[1] *Eng. News-Rec.*, 1921; **87**, 516.

Eddy's table, on which the foregoing conclusion is based, is as follows:

TABLE 38.—EFFECT OF CURVATURE UPON VALUE OF n IN OPEN CHANNELS, CALCULATED FROM EXPERIMENTAL DATA

Channel (concrete lined)	Value of n		Increase due to curvature	Radii of curves, feet	Velocity of flow, feet per second
	On tangent	On curves[1]			
Umatilla	0.0135 to 0.0137	0.0176 to 0.0184	0.0039 to 0.0049	50	7
		0.0162 to 0.0169	0.0025 to 0.0034	100	7
		0.0160 to 0.0173	0.0023 to 0.0038	250	7
Sulphur Creek	0.0108	0.0140	0.0032	2,865	20
Ridenbaugh	0.0121	0.0145	0.0024	955	3.7
North Canal (Central Oregon)	0.0177	0.0202	0.0025	410 and 383	3.0
	0.0176	0.0205	0.0029		2.9
	0.0192	0.0222	0.0030		2.1

[1] Computed.

Experiments by Scobey in 1924 upon the 60-in. reinforced-concrete pipe aqueduct of Tulsa, Okla., are of some significance in this connection, and may be considered as confirming the conclusions above, at least in part. No direct comparison is possible, partly because the aqueduct is a closed pipe under pressure, and partly because the changes in direction are angular bends, not curves. These bends were formed by butting together two sections of straight pipe at the angle required, and pouring concrete over a reinforcing cage at the joint, smoothing off the interior surface to a sharp curve at the outside of the bend. The deflections ranged from less than 5 deg. to about 28 deg.

Two sections were tested, one 80,898 ft. long, nearly straight, (containing 6 bends with a total deflection of 52.3 deg.), the other 34,788 ft. long, containing 29 bends with a total deflection of 455.7 deg. The velocity was 2.25 ft. per second. The values of n in the two sections were 0.0107 and 0.0111, showing an increase of 0.0004 for the section containing the greater changes in direction. In this section the average distance between bends was 1,200 ft. Assuming the effect of the bends in causing disturbed flow conditions to extend 100 ft. below the bends, one-twelfth of the total length of the section was so affected, and the increased loss in this portion would correspond to an increase in n of 12 times 0.0004, or 0.0048,[1] a figure comparable to those tabulated in Table 38.

[1] Based on the assumption that loss of head varies directly with n, which is nearly correct for the small variations here considered.

In his discussion of the Tulsa experiments, Scobey has computed the average loss of head per degree of deflection as 0.00138 ft., and the average per bend, irrespective of the deflection, as 0.0226 ft., for a velocity of 2.25 ft. per second. The extent to which these figures are applicable to other situations, particularly to flow in open channels, is doubtful, but the significant data are so few that nothing which may be helpful should be left out of consideration.

91. "Banking" on Curves.—In some cases, the banking or superelevation of water surface along the outer wall of a curved channel may be an item of considerable importance. This is especially the case with stream channels or open flumes which should not overflow, or flat-topped conduits in which it is desirable that the water should not touch the roof.

The excess in elevation of the water at the outer bank may be computed approximately by the formula

$$E = \frac{v^2 b}{gr}$$

in which E represents the difference in elevation of water surface at the two banks, v is the average velocity in the cross-section, b is the breadth of the channel or stream, and r is the radius of curvature on the center line of the channel.

It has been found in some cases that the actual difference in elevation is slightly greater than would be given by this formula.

NON-UNIFORM FLOW

Conditions of steady non-uniform flow exist when a constant quantity of water flows with variable cross-sections, slopes, and velocities. The surface of the water, therefore, is not parallel to the invert of the conduit. This condition always exists at points of changing equilibrium, such as at and near changes in grade and in cross-section, and above obstructions or free outlets.[1] Typical examples of non-uniform flow are shown in Figs. 46 to 48.

[1] HINDS, JULIAN: The Hydraulic Jump and Critical Depths in the Design of Hydraulic Structures. *Eng. News-Rec.*, 1920; **85**, 1034.

BABBITT, H. E.: Non-uniform Flow and Significance of Dropdown curve in Conduits. *Eng. News-Rec.*, 1922; **89**, 1067.

HILL, C. D.: Application of the Dropdown Curve in Chicago Sewers. *Eng. News-Rec.*, 1923; **90**, 707.

HUSTED, ALVA G.: New Method of Computing Backwater and Dropdown Curves. *Eng. News-Rec.*, 1924; **92**, 719.

WOODWARD, SHERMAN M.: Theory of the Hydraulic Jump and Backwater Curves. *Tech. Rep.*, Part III, Miami Conservancy District.

92. General Equation for Non-uniform Flow.—Assume a reach of conduit short enough so that the loss due to friction may be computed with sufficient accuracy by one of the formulas for uniform flow, such as Manning's formula, making use of the mean hydraulic radii and velocities. Then the average velocity in the reach would be

$$v = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

Taking $v = \frac{1}{2}(v_1 + v_2)$ or the average between the velocities at the ends of the reach,

$$S = \frac{n^2(v_1 + v_2)^2}{8.83 R^{\frac{4}{3}}}$$

Since the total fall in the water surface h is equal to the sum of the frictional loss and the difference in the velocity heads,

$$h = xS + \frac{v_2^2}{2g} - \frac{v_1^2}{2g}$$

The same expression for the drop in water surface is applicable on the basis of any formula for frictional loss.

If d_1 and d_2 represent the depths of water at the two ends of the reach, and i the inclination of the bottom, then

$$h = xi + (d_1 - d_2)$$

Equating the values of h,

$$x = \left\{ d_1 + \frac{v_1^2}{2g} - \left(d_2 + \frac{v_2^2}{2g} \right) \right\} \div (S - i)$$

From this expression the distance x between any two sections of the stream in which the change in depth is $d_1 - d_2$ can be computed approximately.

The foregoing expressions are general and may be applied to any case of steady non-uniform flow, within the limits of the accuracy of the assumptions.

93. Critical Depth.—In the case of free discharge, illustrated in Fig. 46, or of a decided increase in inclination of the channel, as shown in Fig. 48, the depth of flow at the outlet, or at the break in grade, will be definitely fixed by the rate of discharge, for any given conduit. The case is analogous to the discharge of a weir. This depth is called the *critical depth*, designated as d_c, and is the

depth for which $d + \frac{v^2}{2g}$ is a minimum. Then for a rectangular section $d_c + \frac{Q^2}{2gb^2d_c^2}$ is a minimum, whence

$$d_c = \frac{Q^{2/3}*}{g^{1/3}b^{2/3}} \text{ or } d_c^3 = \frac{Q^2}{gb^2}$$

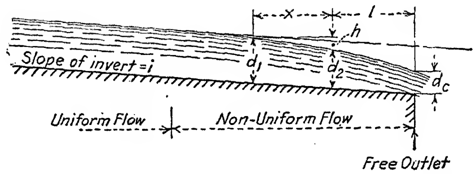


FIG. 46.—Draw-down curve, free discharge.

In this expression, for a given depth d_c, Q varies directly with b, as would be expected for the rectangular section. It is, therefore, practicable to construct a table giving values of Q per foot of width for various values of d_c. Such values are given in Table 39.

TABLE 39.—THEORETICAL DISCHARGE PER FOOT OF WIDTH FOR VARIOUS VALUES OF "CRITICAL DEPTH" IN RECTANGULAR CHANNELS

$Q^{2/3} = d_c g^{1/3} \therefore Q = (d_c)^{3/2} g^{1/2}$ (slide-rule computations))

Depth, d_c ft.	Discharge in cubic feet per second									
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.0	0.18	0.51	0.94	1.44	2.00	2.64	3.33	4.07	4.86
1	5.70	6.58	7.50	8.45	9.43	10.4	11.5	12.6	13.7	14.8
2	16.0	17.3	18.5	19.8	21.1	22.4	23.8	25.2	26.6	28.0
3	29.5	31.0	32.5	34.0	35.1	37.1	38.7	40.4	42.0	43.7
4	45.4	47.1	48.8	50.6	52.4	54.2	56.0	57.8	59.7	61.6
5	63.5	65.5	67.4	69.4	71.3	73.3	75.3	77.3	79.4	81.5
6	83.5	75.6	87.8	89.9	92.0	94.2	96.4	98.5	100.6	102.8
7	105.0	107.2	109.5	111.8	114	116	119	121	123	126
8	128	131	133	136	138	140	143	146	148	151
9	153	156	158	161	164	166	169	171	174	176
10	179	182	185	188	190	193	196	199	202	205

Similar computations of the critical depths corresponding to various rates of discharge for other forms of cross section can be made, but they are likely to be very complex, and it is usually simpler to obtain the values by successive approximations. Prof. H. E. Babbitt[1] has done this for circular conduits and obtained the values given in Table 40.

* For this case $v = \sqrt{gd}$ and $v^2/2g = d/2$ (velocity head = ½ depth).

[1] *Eng. News.-Rec.*, 1922; **89**, 1069.

All these figures are based upon theoretical computations. No experimental determinations of their correctness have been made. It is believed, however, that actual values are not likely to vary from those computed in this way, by more than 10 per cent.

TABLE 40.—THEORETICAL VALUES OF "CRITICAL DEPTH" IN FEET, IN CIRCULAR CONDUITS FOR VARIOUS RATES OF DISCHARGE

(H. E. Babbitt in *Eng. News-Rec.*, 1922; **89**, 1069)

Rate of discharge, cubic feet per second	Diameter of conduit, feet									
	1	2	3	4	5	6	7	8	9	10
0.2	0.20									
0.4	0.27									
0.6	0.33									
0.8	0.38									
1.0	0.41	0.33								
1.5	0.52									
2	0.60	0.48								
3	0.73									
4	0.84									
5	....	0.80	0.66	0.64						
10	....	1.12	0.99	0.92	0.87	0.80				
15	....	1.42	1.23	1.12						
20	....	1.64	1.44	1.28	1.19					
30	....	1.80	1.80	1.60	1.54					
40	....	1.94	2.07							
50	....	....	2.28	2.08	1.98	1.89	1.79	1.72	1.71	1.60
100	....	....	2.88	3.08	2.85	2.67	2.56	2.48	2.39	2.25
150	....	....	....	3.64	3.73					
200	....	....	....	3.92	3.98	3.87	3.68	3.56		
300	....	....	....	....	4.65	4.80	4.55	4.40	4.14	4.15
400	....	....	....	....	4.86	5.44	5.22	5.04		
500	....	....	....	....	....	5.69	5.81	5.68	5.49	5.30
600	....	....	....	....	....	....	6.37	6.40		
700	....	....	....	....	....	....	6.55	6.68	6.53	6.35
800	....	....	....	....	....	....	....	7.04		
900										
1,000	....	....	....	....	....	....	6.79	7.48	7.74	7.70
1,200	....	....	....	....	....	....	....	....	8.24	8.25
1,400	....	....	....	....	....	....	....	....	8.55	8.75
1,600	....	....	....	....	....	....	....	....	....	9.25

94. Drawdown.—The transition from a condition of uniform flow in the conduit to the discharge at a free outlet or the drop at

a chute is accomplished by a gradual lowering of the surface similar to the surface curve above a weir. If the conduit is on a flat slope and the normal velocity is low, the difference between the normal depth and the "critical depth" will be considerable, the drawdown will be of consequence and the drawdown curve will extend upstream for a material distance. In such a case it may sometimes be possible to make a saving by reducing the size of the conduit and eliminating the drawdown, or by lowering the roof while leaving the width unchanged. If, however, the conduit is on such a steep slope that the velocity is high and the critical depth but little less than the normal depth, no material reduction in size of conduit can be made, and the drawdown will not be significant. In those cases, too, where there is possibility that the conduit may flow full under pressure at times, reduction in section for drawdown would be undesirable.

The distance from the control section (the point of "critical depth"), to which the drawdown extends, can be computed approximately by successive steps, utilizing the general equation for non-uniform flow. For example, take the case of a rectangular conduit 10 ft. wide, with a slope of 0.001, with $n = 0.015$ and $Q = 250$ c.f.s., discharging freely as in Figs. 46 and 48. Using Manning's formula, we find, for $S = 0.001$, when $d = 4.0$, $Q = 214$, and when $d = 5.0$, $Q = 288$; and by interpolating, when $Q = 250$, $d = 4.5$, which will be the depth of flow when the effect of the drawdown is not felt. From Table 39, the critical depth d_c will be 2.7 ft. The drawdown curve will be that portion of the water surface between depths of 4.5 and 2.7 ft. Its computation is shown in the following tabulation:

d	A	p	R	v	$\frac{v^2}{2g}$	$d+\frac{v^2}{2g}$	Average R	Average v	nv	S	$S-i$	Change in $d+\frac{v^2}{2g}$	x
2.7	27	15.4	1.75	9.28	1.34	4.04							
							1.81	8.81	0.132	0.0035	0.0025	0.04	16
3.0	30	16.0	1.87	8.33	1.08	4.08							
							1.95	7.84	0.118	0.0025	0.0015	0.16	107
3.4	34	16.8	2.02	7.35	0.84	4.24							
							2.09	6.98	0.105	0.0019	0.0009	0.24	266
3.8	38	17.6	2.16	6.61	0.68	4.48							
							2.23	6.32	0.0095	0.0014	0.0004	0.27	675
4.2	42	18.4	2.28	5.97	0.55	4.75							
							2.33	5.76	0.00865	0.0011	0.0001	0.23	2,300
4.5	45	19.0	2.37	5.56	0.48	4.98							
Total length of drawdown													3,364

95. Backwater.—The surface curve assumed by the water when backed up by a dam or other obstruction is called the backwater curve (Fig. 47). It may be required to determine the amount by which the depth is increased at specified points, or the distance upstream to which the effect of backwater can be detected.

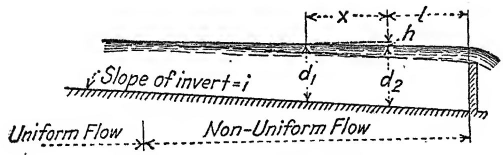


FIG. 47.—Backwater curve.

For example, take a conduit of the same dimensions and character as in the example under "drawdown," but where the discharge is into a body of water with its surface 7.0 ft. above the bottom of the conduit. The procedure is exactly the same as in the example referred to, but the increments of depth are negative, and the value of $S - i$ is also negative. The computation for this case is shown on page 152.

96. Chute.—A chute is a channel with so steep a grade that uniform flow can take place at a depth less than the "critical depth" (Fig. 48). The computation of flow in a chute, in so far as such flow is uniform, is accomplished by the use of the

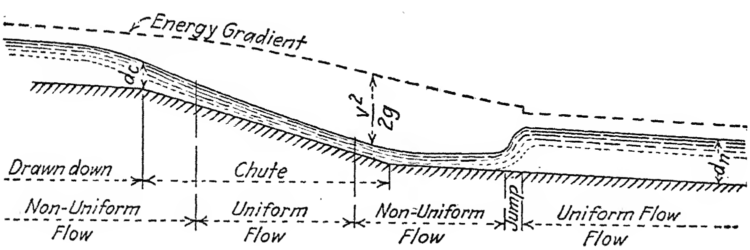


FIG. 48.—Draw down chute, and hydraulic jump.

ordinary formulas, such as Kutter's or Manning's, although the applicability of these formulas at very high velocities is doubtful. Chutes are, however, usually short, and the portion of their length in which uniform flow conditions exist is often insignificant. Non-uniform flow obtains at the upper end of the chute, and the "hydraulic jump" may occur at the lower end if conditions beyond the chute are such as to produce it.

COMPUTATION OF BACKWATER CURVE

d	A	p	R	v	$\frac{v^2}{2g}$	$d + \frac{v^2}{2g}$	*Average* R	*Average* v	nv	S	$S - i$	Change in $d + \frac{v^2}{2g}$	x
7.0	70	24	2.91	3.57	0.20	7.20							
							2.87	3.71	0.0557	0.00033	−0.00067	−0.47	700
6.5	65	23	2.82	3.85	0.23	6.73							
							2.77	4.01	0.0603	0.00043	−0.00057	−0.46	807
6.0	60	22	2.72	4.17	0.27	6.27							
							2.67	4.36	0.0655	0.00054	−0.00046	−0.45	978
5.5	55	21	2.62	4.55	0.32	5.82							
							2.56	4.77	0.0717	0.00067	−0.00033	−0.43	1,300
5.0	50	20	2.50	5.00	0.39	5.39							
							2.44	5.28	0.0793	0.00086	−0.00014	−0.41	2,920
4.5	45	19	2.37	5.56	0.48	4.98							
										Total extent of backwater			6,705 ft.

97. Hydraulic Jump (Fig. 48).—When water moving at a high velocity in a comparatively shallow stream strikes water having a substantial depth, there is likely to be a rise in the water surface of the stream, forming what is called "the hydraulic jump." The jump cannot occur unless the primary depth of the water is less than the critical depth, and when it does occur the water surface rises from a point below to a related point above that representing the critical depth. The depth d_2 of water which will cause the swift-moving stream to form the jump may be computed from the equation

$$\frac{d_1 d_2(d_1 + d_2)}{2} = \frac{q^2}{g} = d_c^3$$

The derivation of this relation is given below.

Let q equal the flow in 1 ft. of width of channel.

Let d_1 and v_1, d_2 and v_2, d_c and v_c be, respectively, the depth and velocity in the channel immediately above the jump, below the jump, and at the point of critical depth. As shown on p. 148

$$d_c^3 = \frac{q^2}{g} \text{ or } q = \sqrt{gd_c^3}$$

but

$$q = v_1 d_1 = v_2 d_2.$$

The change in velocity in passing through the jump is $v_1 - v_2$. The mass of water passing in 1 sec. is wq/g and the change in momentum is

$$\frac{wq}{g}(v_1 - v_2) = \frac{wq}{g}\left(v_1 - v_1\frac{d_1}{d_2}\right) = \frac{wqv_1(d_2 - d_1)}{gd_2}$$

The static pressure upon the cross-section of the stream is $wd_1^2/2$ above the jump, and $wd_2^2/2$ below the jump. The difference is

$$\frac{wd_2^2}{2} - \frac{wd_1^2}{2} = \frac{w}{2}(d_2^2 - d_1^2).$$

The difference in static pressure represents a force acting in the opposite direction to the flow, and this is the force which causes the change in momentum represented by the reduction in velocity from v_1 to v_2.

Therefore

$$\frac{w}{2}(d_2^2 - d_1^2) = \frac{wqv_1}{g}\frac{(d_2 - d_1)}{d_2}$$

Whence

$$d_2^2 + d_2 d_1 = \frac{2qv_1}{g}$$

Substituting $\frac{q}{d_1}$ for v_1,

$$d_2^2 + d_2 d_1 = \frac{2q^2}{gd_1}$$

Whence

$$d_1 d_2 \frac{(d_1 + d_2)}{2} = \frac{q^2}{g}$$

which has been shown to be equal to d_c^3.

For example, take the case of 250 c.f.s. flowing in a rectangular conduit 10 ft. wide, with $n = 0.015$, as before; assume the inclination of the upper section to be 0.05; using Manning's formula (assumed applicable to very high velocities) the depth of flow in the upper section (chute) is fixed at 1.17 ft., which is less than the critical depth of 2.7 ft.

The normal depth of flow in the lower section with its slope of 0.001 is 4.5 ft. Assuming that this is the depth below the jump, the depth above the jump may be computed from the formula above as follows:

$$4.5d_1\frac{(4.5 + d_1)}{2} = \frac{25^2}{32.2}$$

$$d_1^2 + 4.5d_1 = \frac{1{,}250}{144.9} = 8.64$$

$$d_1^2 + 4.5d_1 + 5.06 = 13.70$$

$$d_1 + 2.25 = 3.7$$

$$d_1 = 1.45$$

When the flow reaches the bottom of the chute, the flatte slope retards the flow and the depth increases. Assuming that the chute has sufficient length for the depth of flow to decrease from the critical depth at the top of the chute to approximately the normal depth of 1.17 ft. at the foot of the chute, the flow must continue in the lower section until the retardation has caused an increase in the depth of flow from 1.17 to 1.45 ft. The distance over which the flow must continue before the jump will occur is shown by the following computation.

COMPUTATION FOR HYDRAULIC JUMP

Depth d	Area A	p	R	v	$\frac{v^2}{2g}$	$d + \frac{v^2}{2g}$	Average R	Average v	nv	S	$S - i$	Change in $d + \frac{v^2}{2g}$	x
1.17	11.7	12.34	0.95	21.35	7.08	8.25							
							0.96	21.09	0.316	0.0479	0.0469	0.31	6.6
1.2	12.0	12.40	0.97	20.83	6.74	7.94							
							1.00	20.03	0.300	0.0408	0.0398	0.89	22.4
1.3	13.0	12.60	1.03	19.23	5.75	7.05							
							1.06	18.54	0.278	0.0324	0.0314	0.70	22.3
1.4	14.0	12.80	1.09	17.85	4.95	6.35							
							1.105	17.55	0.263	0.0274	0.0264	0.29	11.0
1.45	14.5	12.90	1.12	17.25	4.61	6.06							
					Total distance below chute								62.3

The jump will be located about 62 ft. below the end of the chute. It is to be noted that under some conditions of channel cross-section and slope, the length of the lower section may be insufficient for the water surface to rise to the required depth. It should also be noted that if the cross-section of the lower section differs from that of the chute so that the critical depth is below the depth of flow after it reaches the lower section, no jump will occur.

METHODS OF MEASURING SEWAGE

98. Weirs.—One of the most accurate methods of measuring water is by the use of a weir, provided the conditions under which the coefficients of discharge of given types of weirs were determined are approximately reproduced in the gagings.

The most common types of weirs are the rectangular, the V-shaped, and the trapezoidal weir.

The General Weir Formula may be expressed by the equation $Q = clH^{3/2}$. To this form all the equations in use may be reduced, but it is better practice, in view of the several methods of correcting for the velocity of approach followed by the various experimenters, to use their forms of equation.

The Francis Weir Formula:

Let H = the observed head corrected to include the effect of the velocity of approach

h = the observed head upon the crest of weir, being the difference in elevation in feet between the top of the crest and the surface of the water in the channel, at a point upstream, which should, if possible, be taken just beyond the beginning of the surface curve

h_v = the head due to the mean velocity of approach $= V_a^2/2g$

For contracted weirs, neglecting velocity of approach,

$$Q = 3.33(l - 0.1Nh)h^{3/2}$$

Note.—The use of h instead H in the factor $(1 - 0.1\,NH)$ used in correcting for end contractions is as precise as ordinary practice warrants.

For contracted weirs, head corrected for velocity of approach:

$$Q = 3.33(l - 0.1NH)\ \ (h + h_v)^{3/2} - h_v^{3/2}$$

For suppressed weirs, neglecting velocity of approach,

$$Q = 3.33lh^{3/2}$$

For suppressed weirs, head corrected for velocity of approach:

$$Q = 3.33l\,[(h + h_v)^{3/2} - h_v^{3/2}]$$

The Fteley and Stearns formula:

$$Q = 3.31lH^{3/2} - 0.007l$$
$$H = (h + 1.50h_v) \text{ for suppressed weirs}$$
$$H = (h + 2.05\,h_v) \text{ for contracted weirs}$$

For contracted weir use $(l - 0.1NH)$ instead of l.

The King formula,[1] based upon the experiments of Francis, Fteley and Stearns, and Basin, is

$$Q = 3.34lH^{1.47}\left(1 + 0.56\frac{H^2}{d^2}\right)$$

where $d = A/l$, A being the area of the channel of approach.

For contracted weir use $(l - 0.1NH)$ instead of l.

Lyman's Diagrams and Tables[2] are based upon the experiments of Francis, Fteley and Stearns, and Bazin, and at the hydraulic laboratories of Cornell University and the University of Utah, and are in convenient form for use.

The Francis formulas are strictly applicable only to vertical sharp-crested rectangular weirs with complete contractions and with free overfall and

when the head (H) is not greater than one-third the length (l);
when the head is not less than 0.5 ft. nor more than 2 ft.;
when the velocity of approach is 1 ft. per second or less;
when the height of the weir is at least three times the head.

In all probability the formulas are usable with higher heads than 2 ft., but not much lower than 0.5 ft., as shown by Fteley and Stearns' experiments.

Rehbock's Formula, first published in Germany in 1911, and revised in 1912, is as follows:[3]

$$Q = \tfrac{2}{3}\mu\sqrt{2g}lh^{3/2}$$

where

$$\mu = 0.605 + \frac{1}{320h - 3} + 0.08\,\frac{h}{Z}$$

where

Z = height of crest of weir above bottom of channel.

[1] KING: "Handbook of Hydraulics," 2nd Ed., 87.

[2] LYMAN, R. R.: *Trans.* Am. Soc. C. E., 1914; **72,** 1189.

[3] In lectures at Massachusetts Institute of Technology (1929) Rehbock gave

$$\mu = 0.6035 + 0.0813\,\frac{h + 0.0033}{Z}$$

and also

$$Q = \left(3.223 + \frac{1}{57h} + 0.427\,\frac{h}{Z}\right)lh^{3/2}$$

This formula is based upon numerous experiments made on sharp-crested, fully aerated weirs without end contractions, for values of

$$Z = 0.41, 0.66, 0.82, \text{ and } 1.64 \text{ ft.}$$
$$h = 0.03 \text{ to } 0.59 \text{ and } 0.75 \text{ ft.}$$
$$l = 1.64 \text{ ft.}$$

He extended the curves for μ beyond the range of his actual experiments by the law of hydraulic similitude to include values of

$$Z = 0.33 \text{ to } 6.56 \text{ ft.}$$
$$h = 0.03 \text{ to } 3.28 \text{ ft.}$$
$$\frac{h}{Z} = 0 \text{ to } 1.0 \text{ (preferably not over 0.8)}$$

Professor Schoder[1] states that Rehbock's formula gives values which agree more closely with a large mass of recent experimental data than the formulas of Fteley and Stearns, Bazin, and Francis, the error increasing with h and with Z. Francis' formula checked closely at low heads and low velocities of approach. Rehbock's formula, while comparatively unknown in this county, is largely used in Germany.

Schoder and Turner[1] have made a comprehensive study of the accuracy of various formulas for determining the discharge over sharp-crested weirs and find an important source of error in such formulas to be in their failure to properly allow for velocity of approach and in unequal horizontal distribution of velocities. They suggest a formula which includes velocity head of approach for the section above and below the elevation of the weir crest, considered separately.

99. Triangular Weirs.—The theoretic discharge of the triangular weir is given by the equation,

$$Q = \tfrac{4}{15}[B(2g)^{\frac{1}{2}}H^{\frac{3}{2}}]$$

in which

B = length of base of triangle at level of H or water surface
H = head over angle of the weir notch in feet

Experiments on flow over triangular weirs in which the angle is a right angle, which is the form most commonly used in prac-

[1] SCHODER, ERNEST W., and KENNETH B. TURNER: Precise Weir Measurements. *Trans.*, Am. Soc. C. E. 1929; **93**, 999.

tice, have been made by Thomson; by Barr; at the University of Michigan; and at the Massachusetts Institute of Technology. Summarized results of the first three sets of experiments are given by King in his "Handbook of Hydraulics," 1st Ed., p. 86. Thomson's experiments covered a range of heads from about 0.15 to 0.60 ft.; Barr's from about 0.15 to 0.85; and the University of Michigan's, from about 0.15 to 1.8. Their results may fairly be expressed by the following equations.

Thomson's experiments, $Q = 2.54H^{5/2}$

Barr's experiments, $Q = 2.48H^{2.48}$

University of Michigan's experiments, $Q = 2.52H^{2.47}$

Experiments made at the Massachusetts Institute of Technology, under the direction of Professor Dwight Porter, gave for the right-angled notch weir,

$$Q = 2.54H^{5/2}$$

100. Trapezoidal Weirs.—The trapezoidal weir differs from the rectangular type in that the sides are inclined rather than vertical. Usually the sides are given a batter of 1 horizontal in 4 vertical for the reason that at this angle the slope is just about sufficient to offset the effect of end contractions. When this is done the weir is known as the "Cippoletti weir." The general equation of the trapezoidal weir is as follows:

$$Q = \tfrac{2}{3}(2g)^{1/2}lH^{3/2} + \tfrac{4}{15}\,2z(2g)^{1/2}H^{5/2}$$

in which z = the batter of the side or the ratio of the vertical projection to the horizontal projection of the side.

For the Cippoletti weir in which $z = 1/4$, the formula reduces to

$$Q = 3.367lH^{3/2}$$

101. Other Weirs.—For the determination of the discharge over broad-crested weirs and dams having different types of crests, reference may be had to ROBERT E. HORTON'S "Weir Experiments, Coefficients, and Formulas;[1]" King's "Handbook of Hydraulics;" Williams and Hazen's "Hydraulic Tables;" and various books on hydraulics.

[1] Published as Water Supply and Irrigation Paper 200 of the U. S. Geological Survey, 1907.

Side-weirs or spillways parallel to the direction of flow in a channel are not measuring devices. They are utilized as regulating devices, and are discussed in Chap. VI.

102. The Venturi Meter.—The Venturi meter tube is inserted in a line of pipe under pressure and consists of three parts, the inlet cone, in which the diameter of the pipe is gradually reduced, the throat or constricted section, and the outlet cone, in which the diameter increases gradually to that of the pipe in which the meter is inserted. The throat is lined with bronze; its diameter,

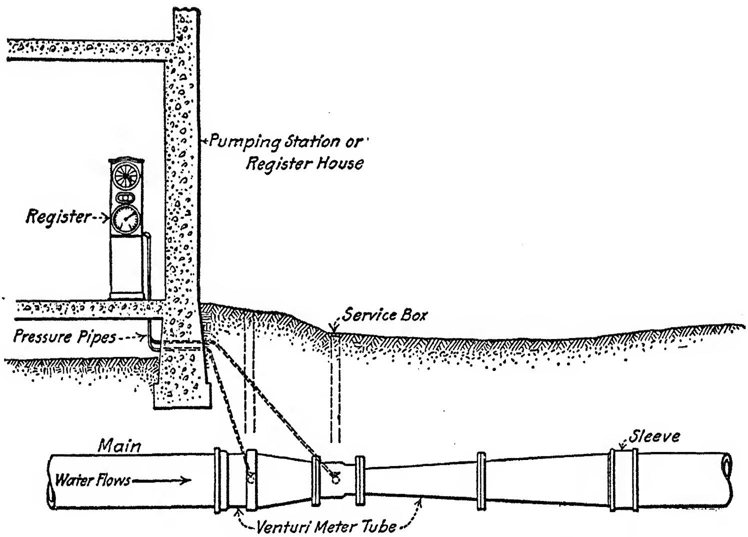


FIG. 49.—Arrangement of venturi meter on pressure pipe.

in standard meter tubes, is from one-third to one-half of the diameter of the pipe; and its length but a few inches, sufficient to allow a suitable pressure chamber or piezometer ring to be inserted in the pipe at this point. The upper or inlet cone has a length of approximately one-fourth that of the lower cone. A piezometer ring is inserted at the upper or large end of the inlet cone and the determination of the quantity of water flowing is based upon the difference in pressures observed or indicated at this point and at the throat of the meter. The general form of the meter is shown in Fig. 49.

The derivation of a formula from which the discharge of the Venturi meter tube is computed may be found in Hughes and

Safford's "Hydraulics," First Edition, p. 116. As written by Herschel, the form of this expression is

$$Q = \frac{A_1A_2}{\sqrt{A_1^2 - A_2^2}} \sqrt{2g(h_1 - h_2)}$$
$$= \frac{A_1A_2}{\sqrt{A_1^2 - A_2^2}} \sqrt{2gH}$$

in which A_1 and A_2 are the areas in square feet at the upstream end and at the throat of the meter, respectively, h_1 and h_2 the pressure heads at the corresponding points, and

$$H = h_1 - h_2$$

Under actual operating conditions, and for standard meter tubes, including allowance for friction, this formula reduces to the form

$$Q = (1.00 \pm 0.02)A_2\sqrt{2gH}$$

The coefficient written (1.00 ± 0.02) is made up of two parts, or $c = c_1c_2$

$$c_1 = \frac{A_1}{\sqrt{A_1^2 - A_2^2}}$$
$$c_2 = \text{coefficient of friction}$$

For standard meter tubes in which the diameter of the throat is between one-third and one-half that of the pipe, the values of c_1 range between 1.0062 and 1.0328, while the friction coefficient c_2 varies from 0.97 to 0.99. Thus the range of values of c is from 0.98 to 1.02, and accordingly c has been written above as (1.00 ± 0.02). Hazen[1] thinks $c = 0.99$ the best value for practical use.

In the Venturi meter used for measuring sewage, at each annular chamber or piezometer ring there should be valves by which the pressure openings can be closed, and these valves may be so designed that in closing a rod is forced through the opening so as to clean out effectually any matter which may have clogged it. When all of these valves have been closed the plates covering the hand holes in the pressure chamber may be removed and the chamber cleaned by flushing with hose or otherwise. Such flushing at short intervals is usually necessary if Venturi

[1] *Eng. News*, 1913; **70**, 199.

meters for sewage are to be maintained in good operating condition.

In order to prevent the interference with the operation of the register by clogging, an oil seal may be inserted in the pressure pipe, between the meter tube and the register. The pressure is transmitted as far as the seal through water in the pressure pipes, and from the seal to the register through oil. Thus it is impossible for any sewage to get into the register and interfere with its proper operation.

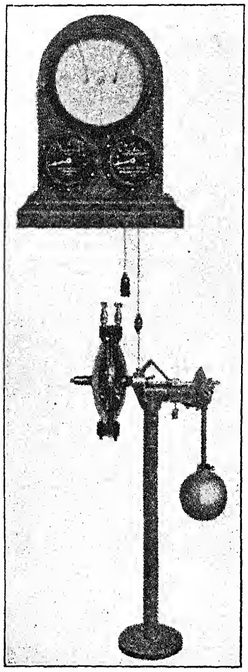


FIG. 50.—Diaphragm pendulum unit for use with Venturi meter.

The diaphragm-pendulum unit of the Builders Iron Foundry makes the use of an oil seal unnecessary in the case of sewage meters, and is applicable to either high or low pressures. This unit is placed below the hydraulic gradient and is connected to an indicating or recording instrument by a cord, as shown in Fig. 50. The diaphragm-pendulum unit consists essentially of a pressure chamber divided vertically by a rubber diaphragm, and a pendulum connected in such a manner as to swing until it balances any pull imposed by the diaphragm. Pressure pipes from the Venturi tube transmit the pressures from the upstream end and the throat to the two sides of the diaphragm, the deflection of which depends upon the difference in pressures and the weight and lever-arms of the pendulum. A cord passing over a pulley connected to the pendulum transmits the measurement of the pressure differential to the recording or indicating instrument. In the case of sewage measurements, "water seals" should be inserted in the pressure pipes between the Venturi tube and the diaphragm pressure chamber. These water seals are simply small vertical tanks, the pressure pipes entering them at the bottom and leaving at the top, and are filled with clean water before the pressure is turned on. Since there is no flow through the pressure pipes, but only a slight movement back and forth as the pressures vary, there is no

possibility of suspended matter passing the seals and reaching the diaphragm chamber.

103. The Venturi Flume.—Since the Venturi type meter is only applicable to closed pipes under pressure, it can be used for measuring sewage only in force mains or inverted siphons.

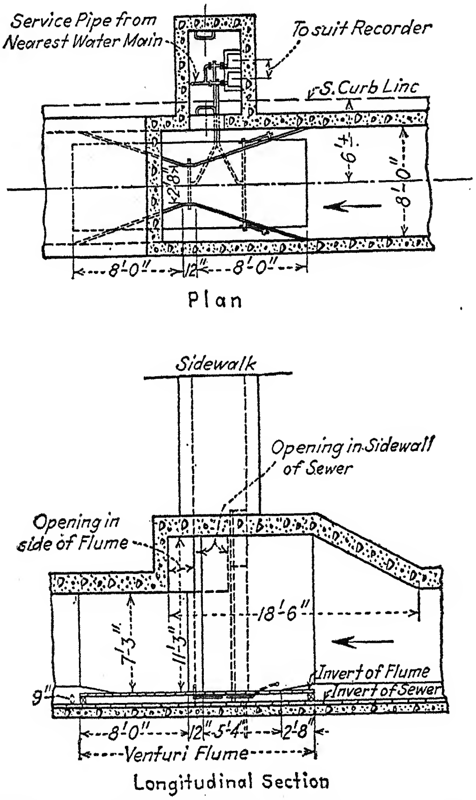


FIG. 51.—Venturi flume inserted in East 41st street sewer, Manhattan, New York.

In recent years, the Venturi principle has been utilized for the measurement of flow of water in open channels by means of the Venturi flume, in which the side walls and in some cases the bottom of the channel are so shaped as to approximate the form of the Venturi tube.

This apparatus has been used principally in connection with the distribution of water in irrigation ditches, but it is being introduced as a device for measuring sewage in several cities. A

flume of this type is used to measure the flow of sewage reaching the Syracuse Sewage Works and another was placed in operation on East Forty-first Street by the Borough of Manhattan in November, 1926. The latter is illustrated by Fig. 51 and is of yellow pine, anchored to the side walls of an 8-by 8-ft. box sewer, the arch of which was raised 4 ft. above the flume to allow for gagings during extremely high flows. The capacity, limited by the recording device to a depth of 10 ft. at the upstream piezometer pipe, is 343 cu. ft. per second. When operating at maximum capacity, the incoming sewer flows with a surcharge of about 2 ft.

From the calibration tests of flumes with throats 1, 1½, 2, 3 and 5 ft. in width, and with discharges ranging from less than 1 to nearly 400 cu. ft. per second, Parshall and Rowher derived the formula for this type of flume

$$Q = cbh_2\sqrt{\frac{2gh}{1 + \frac{9}{49}\left(\frac{h_2}{h_1}\right)^2}}$$

in which Q = discharge in cubic feet per second
b = width of throat in feet
h_2 = head at throat in feet
h_1 = head in converging section in feet
$h = h_1 - h_2$
c = a coefficient, the value of which can be obtained from the equation

$$c = 0.9975 - 0.0175b + \frac{(h - 0.163h_1^{1/2})^2(20 - b)}{8h_1^2}$$

A comparison of 453 actual measurements with the results computed by this formula showed that 55 per cent agreed within 2 per cent; 68 per cent within 3 per cent; and 88 per cent within 5 per cent. The formula may, therefore, be considered sufficiently accurate for most practical purposes.

104. Parshall Measuring Flume.—*Press Bull.* 60 of the Agricultural Experiment Station of the Colorado Agricultural College, dated January, 1925, by Ralph L. Parshall, announced the design of an improved Venturi flume, in which the discharge is a function of the width of the throat and the head in the converging section only, and that these conditions obtain up to depths of submergence (at the end of the converging section) of

nearly 75 per cent of the depth at entrance. The formula for the discharge of this flume is

$$Q = 4bh^{1.522b^{0.026}}$$

Since the throat width is a constant, the discharge can be obtained from a single measurement and it is possible to graduate a tape attached to a float in the gage well so as to show directly the quantity flowing. One hundred fifty-two experiments have been made on this flume at Ft. Collins, with throats 1 to 8 ft. wide, and with discharges from 0.05 to 60 cu. ft. per second; 88 per cent of them agreed with the formula within 3 per cent.

Further data are given in a paper by Parshall.[1] The total loss of head in a 3-ft. flume of this pattern is reported as 0.19 ft. for a discharge of 9.48 c.f.s. In discussing this paper, H. B. Muckleston calls attention to the fact that such a device, where but one head is measured, is really a special form of weir, rather than a true Venturi flume.[2] He also gives a simple form of diagram for the solution of the formula for discharge; and C. E. Carter shows that for the data presented, the formula

$$Q = 3.90bh^{1.58}$$

gives results quite as consistent with those obtained by tests as does the more complex formula presented by Parshall.

105. The use of dyes for measuring the velocity of flow in sewers, particularly in small pipe sewers, is one of the simplest and most successful methods that has been used. Having selected a section of sewer in which the flow is practically steady and uniform, the dye is thrown in at the upper end, and the time of its arrival at the lower end is determined. If a bright-colored dye is used, such as eosin, and a bright plate is suspended horizontally in the sewer at the lower end, the time of appearance and disappearance of the dye at the lower end can be noted with considerable precision, and the mean between these two observed times may be taken as representative of the average time of flow.

106. Chemical and electrical methods of gaging, while of great value in measuring clean water, are not so applicable to sewage measurements on account of the relatively large amounts of foreign matter contained in the sewage. They have been used, however, in measuring the flow through settling basins with apparent success.

[1] *Trans.*, Am. Soc. C. E., 1926; **89**, 841.

[2] Julian Hinds suggests the name "control-section meter."

Floats and current meters are sometimes used for measuring flow of sewage, especially in large conduits. For a discussion of these methods of measuring flow, consult any standard work on hydraulics.

Problems

1. A sewer 800 ft. long with a fall of 2.4 ft. must carry 10 cu. ft. per second when full. What size of circular vitrified-pipe sewer is required? What will be the velocity of flow? What coefficient n should be used in the Kutter formula?

2. In the sewer of Problem 1, a minimum rate of flow of 0.75 cu. ft. per second is expected. Find the depth and velocity under these conditions, according to Kutter's formula.

3. Re-design the same sewer using an egg-shaped section entirely filled and compare the depth and velocity at minimum flow with the results of Problem 2.

4. In an existing sewer system, a 96-in. concrete sewer is found to have been laid on a slope of 0.00085 and its condition is found to warrant the use of $n = 0.013$. What is its capacity and the corresponding velocity of discharge in it when filled? When filled to 0.65 depth? When filled to 0.3 depth? (According to Kutter's formula.)

5. The sewer of Problems 1 and 2 discharges into a concrete trunk sewer ($n = 0.013$) 48 in. in diameter on a slope of 0.002 and which carries a minimum flow of 9 cu. ft. per second above the junction. For topographical reasons it is impracticable to build the crowns of the two sewers coincident in order to avoid surcharging the smaller sewer when the larger is full; it is, however, desired to avoid backing up in the smaller sewer at times of minimum flow in both sewers. At what elevation, above the invert of the trunk sewer, must the invert of the lateral be connected?

6. The following field notes are given for the alignment of a natural stream which is to be replaced by a rubble-lined channel ($n = 0.017$); tangent 325.0 ft., curve 125.0 ft., tangent 480.0 ft., curve 120.0 ft., tangent 615.0 ft., curve 85.0 ft., tangent 1,080 ft., curve 470.0 ft. Assuming that consideration of expected velocities and the radii of the curves leads to the decision to compensate for the extra slope required on curves by the addition of 0.004 to the value of n on the curves, determine a balanced value of n to use throughout the entire length of the proposed channel.

7. A sewer is to be designed where the slopes are such that the velocity when full in the upper stretch will be 3 ft. per second and in the lower stretch 9 ft. per second. Assuming that the flow will just fill the sewer, what is the required drop in the hydraulic gradient to give this increased velocity?

8. Show by a sketch the relative position giving difference in invert elevations of the two sewer sections to meet the conditions of Problem 7 so there will be no surcharge of either sewer section.

9. The compartments of a grit chamber in which the velocity is 1 ft. per second terminate in an outlet channel in which the velocity is 6 ft. per second. The flow from the chamber must pass through gates which reduce the

cross-section of flow at the point of passage to 60 per cent of that available in the grit compartments. Estimate the total loss of head, including the loss of head through the gates and that necessary to produce the increase in velocity in the outlet channel, but neglecting the friction loss.

10. The several compartments of a grit chamber in which the velocity is maintained at about 1 ft. per second converge into a single outlet channel. For what velocity shall the outlet channel be designed so that the loss of head (drop in water surface) involved in increasing the velocity shall not exceed 0.5 ft., out of which an allowance of 0.15 ft. must be made for the losses through gates, etc. ?

11. In passing through a transition section into a large section the velocity in a conduit is reduced from 7 ft. per second to 4 ft. per second. The water surface in the larger section is found to be 0.3 ft. above the hydraulic gradient, as determined by the normal slopes required for friction alone. What is the percentage of recovery of velocity head in the transition from the smaller to the larger section?

12. The influent conduit of the Dayton, Ohio, Imhoff tanks, has a closed rectangular section at one point 2 ft. 9 in. wide by 4 ft. 5¾ in. high with a 4 in. by 4 in. chamfer in each corner. Assuming a slope of 0.001 and a coefficient c of 110 in the Chezy formula, plot a curve for the discharge and velocity when flowing at partial depths and full.

13. Compute the data for the drawdown curve for a rectangular conduit 8 ft. wide with a slope of 0.0005, with $n = 0.015$ and $Q = 150$ c.f.s., discharging freely.

14. The conduit in Problem 13 discharges into a reservoir the water surface of which is 6.0 ft. above the bottom of the conduit. Compute the data for the backwater curve for $Q = 150$ c.f.s.

15. The conduit in Problem 13 has an upper section of the same width and material but with a slope of 0.05; assuming Manning's formula to be applicable to such high velocities, determine depth of flow in this section. Compute distance from point at which the slope changes to 0.0005, to where the jump will occur. Draw a profile indicating depths through both upper and lower sections of the flume and locating the jump.

16. A concrete influent flume of rectangular section discharges 3.0 c.f.s. into a series of three Imhoff tanks through 6 submerged circular orifices spaced 5 ft. on centers and with two feeding each tank. The discharge through each orifice is to be 0.5 c.f.s. and velocity in the flume is 3 ft. per second throughout its length. Determine section of flume between each pair of orifices, diameters of orifices and hydraulic gradient through flume into tanks. $n = 0.012$.

CHAPTER V

DESIGN OF SEWERS

107. Definitions.—A *separate sewer* is a sewer intended to receive domestic sewage and industrial wastes without admixture of surface or storm water.

A *combined sewer* is a sewer intended to receive domestic sewage, industrial wastes, and surface and storm water.

A *drain* is a conduit for carrying off storm water, surface water, and subsoil or ground water, domestic sewage and industrial wastes being excluded.

SEPARATE SEWERS

108. Reasons for Adopting the Separate System.—The construction of a system of separate sewers alone, or with only a partial system of storm drains, has become common practice in small communities. This has been due generally to economic necessity, either real or fancied. The smaller towns frequently consider it impossible to finance an adequate system of combined sewers, and it is often permissible to allow storm water to flow in gutters and natural water courses for many years after the necessity for separate sewers has become pressing. Even where combined sewers would not be prohibitive in cost, it is generally easier to secure funds for the less costly separate sewers. This often leads to the adoption of separate sewers without a thorough study and careful weighing of the comparative merits of the two systems.

In some cases, also, the discharge of sewage mingled with storm water, even during brief overflow periods of storms, is not permissible—a condition which may require the adoption of separate sewers.

109. Maps and Profiles.—In the design of a system of separate sewers, a topographic map is desirable, on a scale of about 200 ft. to the inch, showing contours, brooks, rivers, ponds and streets. The contours should be sufficiently close to allow the designer to plot profiles of streets with reasonable accuracy;

i.e., where the surface slope is 6 per cent or less the map should show contours at 2-ft. intervals; where the surface slope is much greater than this, 5-ft. intervals will usually suffice. Summits in streets should be marked and the elevations given to tenths of a foot, as should also points of depression or "pockets."

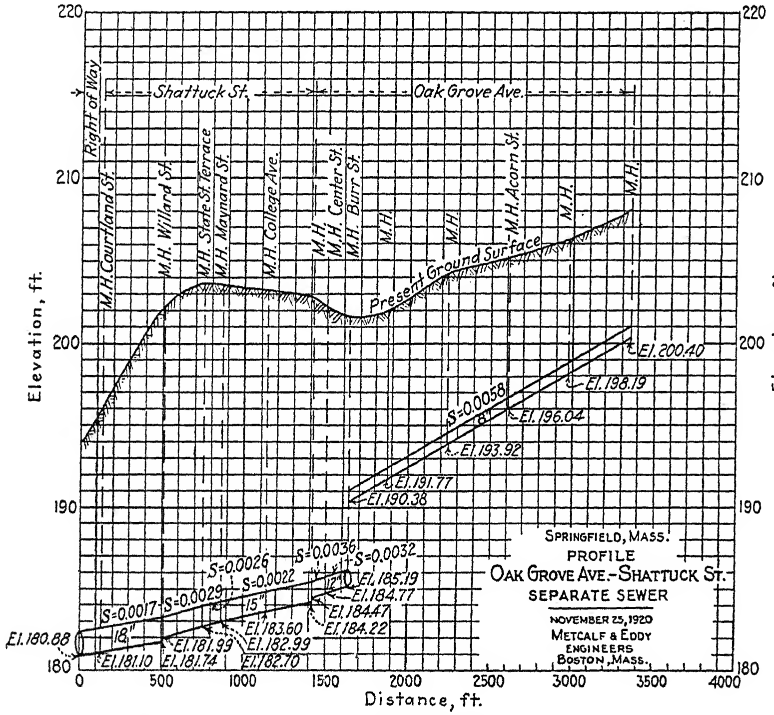


FIG. 53.—Profile for problem on separate sewer design.

Profiles of all or most of the sewers should be drawn as in Fig. 53 and the slopes and sizes of the sewers tentatively determined for preparing the preliminary estimate of cost of the proposed work. This will indicate whether the sewers are everywhere at suitable depths to serve the buildings and at the same time whether drops in the manholes may be advantageous at certain points, or whether the design can be improved in other particulars. A profile of the ground surface along the line of the

main or trunk sewer will generally be desirable in advance of the computations.

110. Basic Design Data.—The steps and basic data used in the design of a separate system may be illustrated by the following example.

The district shown in Fig. 52 is a portion of the Carlisle Brook District of Springfield, Mass. This is a residential district, which, in 1920, was about two-thirds developed with detached houses. This district is so nearly fully developed that the probable future density of population can be estimated with a fair degree of certainty and it is not necessary to make a study and curve such as is shown on Fig. 9.

It is estimated that the future average density of population will be 65 persons per acre. The maximum hourly rate of flow of sewage is estimated at 250 gal. per capita per day, including 30 gal. per capita per day allowance for storm-water run off.

The maximum rate of ground water infiltration to the sewers, to be provided for, is 2,000 gal. per acre per day. This should not be confused with the allowance for storm water run-off above mentioned.

The minimum size of sewer is to be 8 in. The minimum velocity of flow in the sewer when full is to be 2 ft. per second (the more desirable limit of 2.5 ft. is impracticable because of the prohibitive expense that would result from its adoption.)

The minimum depth below the street surface to the top of sewers will be 7 ft.

111. Design of a Separate Sewer System.—After compiling the basic data and determining the minimum allowable size of sewer and minimum velocity of flow in the full sewer, the procedure of design is as follows:

First.—Draw a line to represent the sewer in each street or alley to be served. Near the line place an arrow pointing in the direction in which the sewage is to flow. Except in special cases the sewer should slope with the surface of the street. It is usually more economical to plan the system so that the sewage from any street will reach the point of disposal by the most direct and consequently the shortest route.

If the lines are properly drawn, the system will often resemble a tree and its branches, all the laterals connecting with the submains and these in turn with the main or trunk sewer, which leads to the point of discharge.

Second.—Locate the manholes, giving each an identification number. Manholes should generally be placed at all angles or bends, changes in grade, at all junctions of street sewers, at the upper ends of all laterals and at intermediate points where the distance exceeds 400 ft.

Third.—Sketch the limits of the tributary areas for each lateral, unless a single lateral will be required to accommodate an area larger than can be served by the minimum size of sewer with the minimum slope, in which case a further subdivision must be made. Where the streets are laid out, the limits may be assumed as being midway between them. If the street layout is not shown on the plan, the limits of the different areas cannot be determined as closely and the topography must serve as a guide.

Fourth.—Measure the acreages of the several tributary areas. For this a planimeter will give results with sufficient accuracy.

Fifth.—Prepare a table, like Table 41, with columns for the different steps in the computation and a line for each section of sewer between manholes. This is the most concise, time-saving method and keeps the results in better shape for the subsequent use of the draftsman. Use column 1 for numbering the lines of the table, for ready reference. Determine by inspection the manhole which is farthest from the point of discharge and enter its indentification number in the first line of column 2, and the number corresponding to the manhole next on the line toward the trunk sewer in column 3. Enter the name of the street or alley in column 4, the length between manholes in column 5, and the area in acres to be drained by the sewer at a point just above the lower manhole in column 6. On the next line enter the corresponding data for the next stretch of sewer, and in column 7 enter the sum of the areas listed in column 6. The area in column 7 is the basis for computing the required capacity of the sewer. Enter the data for each section of sewer in the above manner, following the line down to the point of discharge, including the trunk or main sewer.

Enter in column 8 the rate of flow in the sewer, which is equal to the maximum per capita rate of sewage flow multiplied by the assumed future density multiplied by the area shown in column 7.

In column 9 enter the rate of ground water flow, which is equal to the rate of flow per acre to be provided for, multiplied by the area in column 7.

TABLE 41.—DATA AND COMPUTATIONS FOR A SEPARATE SEWER SYSTEM

Line	From manhole No.	To manhole No.	Location, street	Length, ft.	Area, acres		Sewage, million gals. daily[1]	Ground water at 2,000 gal. per acre daily	Total maximum flow, sewage and ground water		Size of sewer, internal diameter in in.	Slope, decimal of a ft. per foot	Velocity, ft. per second	Capacity, cu. ft. per second	Surface elevation, upper end	Invert elevation	
					Increment	Total			Million gal. daily	Cu. ft. per sec.						Upper end	Lower end
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1	57	58	Oak Grove Ave.	380	......	......	......	......	......	......	8	0.0058	2.0	0.7	208.2	200.40	198.19
2	58	59	Oak Grove Ave.	370	......	......	......	......	......	......	8	0.0058	2.0	0.7	206.4	198.19	196.04
3	59	61	Oak Grove Ave.	365	......	......	......	......	......	......	8	0.0058	2.0	0.7	205.2	196.04	193.92
4	61	62	Oak Grove Ave	370	......	......	......	......	......	......	8	0.0058	2.0	0.7	204.3	193.92	191.77
5	62	11	Oak Grove Ave.	240	......	12.07	0.196	0.024	0.220	0.34	8	0.0058	2.0	0.7	202.0	191.77	190.38
6	11	12	Oak Grove Ave.	130	35.18	47.25	0.767	0.095	0.862	1.33	12	0.0032	2.0	1.55	201.6	185.19	184.77
7	12	13	Oak Grove Ave.	82	11.80	59.05	0.960	0.118	1.078	1.67	12	0.0036	2.2	1.70	202.1	184.77	184.47
8	13	14	Shattuck	280	5.30	64.35	1.046	0.129	1.175	1.82	15	0.0022	2.0	2.60	202.8	184.22	183.60
9	14	15	Shattuck	275	19.23	83.58	1.360	0.167	1.527	2.36	15	0.0022	2.0	2.60	203.2	183.60	182.99
10	15	16	Shattuck	113	12.14	95.72	1.555	0.192	1.747	2.70	15	0.0026	2.0	2.70	203.6	182.99	182.70
11	16	17	Shattuck	245	2.75	98.47	1.600	0.197	1.797	2.78	15	0.0029	2.3	2.80	203.7	182.70	181.99
12	17	18	Shattuck	375	12.43	110.90	1.800	0.222	2.022	3.13	18	0.0017	2.0	3.55	202.3	181.74	181.10
13	18	19	Right of way	130	4.22	115.12	1.871	0.231	2.101	3.25	18	0.0017	2.0	3.55	196.0	181.10	180.88

[1] Based upon 250 gal. per capita daily and 65 persons per acre. Since the capacity of the minimum size of sewer with a minimum velocity of 2 ft. per second is 0.7 cu. ft. per second, equivalent to the maximum rate of discharge from 24.6 acres, since $\frac{24.6 \times (65 \times 250 + 2{,}000) \times 1.55}{1{,}000{,}000} = 0.7$, all laterals will be 8 in. diameter (the minimum) as no lateral is to drain an area exceeding 24.6 acres.

Column 10 contains the sums of the figures in columns 8 and 9, in million gallons per day, and the figures in column 11 represent this quantity converted into cubic feet per second, the more convenient way of expressing the capacity of sewers and the units for which most diagrams and tables for determining the carrying capacity of sewers are given.

The designer is now ready to prepare profiles similar to Fig. 53, and with their aid to select the sizes and slopes of sewers required to carry the computed quantities.

In column 12 is the required size of sewer; column 13, the slope; in column 14 is the velocity when the sewer is full, while in column 15 is the carrying capacity.

In column 16 is the elevation of the ground surface at the manhole corresponding to the identification number in column 2.

Columns 17 and 18 contain the invert elevations of the upper and lower ends, respectively, of each stretch of sewer.

If the computations for the entire system are carried out in all the detail above outlined, it may not be necessary to construct profiles for the preliminary study, but it is advisable to make profiles of the main sewers and the more important laterals as a check upon the work. For construction purposes, a plan and profile should be prepared for each sewer from data obtained by field observations and survey, showing the ground surface, the depth and location of existing cellars or basements, the proposed sewer, its slope and size in figures, and the invert elevation at each manhole, as well as the size and elevation of the sewer into which the sewer under consideration is to discharge. The scale to be used in preparing the profiles will depend upon the number of obstacles to construction, and hence the amount of detail required. In city work, scales of 20 to 40 ft. per inch horizontally and 2 to 4 ft. per inch vertically are commonly used. The profile should be drawn either directly above or below the location plan. The plan should be of the same scale as the horizontal scale of the profile and should show all structures both above and below ground surface, which may influence the choice of location for the sewer or which may affect the construction operations.

From the figures in the tabular computation, depths of cut may be computed and estimates of cost prepared.

112. Minimum Size of Pipe Sewers.—The adoption of a minimum size is necessary because experience has shown that some comparatively large objects such as scrubbing brushes, for

instance, do sometimes get into sewers, and that stoppages resulting from them are less likely if sewers smaller than 8 in. (or perhaps 6 in.) are not used. Obviously, the smallest sewer should be at least as large as, and preferably larger than, the building connections in general use, so that articles which pass the building connections will as readily pass the sewer.

Engineers are not in entire agreement upon the most advantageous size of building connections. The most common size is probably 6 in., but 5 and 4 in. are frequent. It is good practice to use one size for sewer connections and another size for drain connections, thus reducing the likelihood of either being connected to the wrong conduit in the street.

113. Drop in Manholes.—In general, losses of head in well-constructed manholes are insignificant, although under some circumstances they may be noticeable. Years ago, when a manhole was usually a well with a sewer entering at one side and leaving at the other, and frequently with a sump extending some distance below the invert of the sewer (for the ostensible purpose of collecting grit for removal), losses of head in manholes were considerable and drops should have been and usually were provided. At present, manholes are built with channels formed in them, connecting the ends of the entering and leaving sewers, and there is no material enlargement of the stream of sewage until its surface rises above the bench.

Drops in the invert line should generally be introduced where the inside vertical height is increased, to provide an increased depth of flow in passing downstream, thus keeping the crowns of sewers up to the hydraulic gradient when full. Other drops may be introduced to provide for maintaining the sewer at the proper depth to drain adjacent buildings or for the purpose of using up excess fall. Often, however, there is no excess fall, the slopes required to maintain the flow result in adequate depth below the surface, and it is possible and desirable to carry the sewer through the manholes with only the fall due to the slope of the sewer.

114. Low-flow Conditions.—Sewers with flat slopes are often required to avoid excessive cut, provide a minimum depth of cover or to satisfy local conditions, such as flat surface slopes or small total fall available.

Even if a suitable minimum slope is adopted, it may occasionally be found necessary to flush lateral sewers near their upper ends, as there is often insufficient flow to carry the solids along

even after complete development. Farther down, the laterals and submains will usually not require flushing since they will ordinarily not be installed until a considerable development has taken place.

Where a long main sewer is required to provide an outlet for an isolated development, but is designed of sufficient size to accommodate future development along its route, the conditions of flow should be investigated to ascertain whether costly flushing and cleaning are likely to be required. In separate sewers, the minimum flow will occur during the early morning hours and will consist largely of ground water infiltration and leakage from fixtures. Thus it will be nearly clear water, and objectionable deposits are not likely to be formed even if the velocities at such times are very low. On the other hand, if the velocities of flow are low during the daylight hours when the higher and average rates of discharge take place, objectionable deposits may be formed; and rather than allow this it may be wise to redesign the sewer to obtain better velocities, possibly even at the expense of installation of a temporary pumping station. Examination of the design for determining low flow conditions should be made by the method outlined in Section 124 later on in this chapter.

115. Alternate Projects.—It may sometimes be found that two or more alternate arrangements are practical for some of the sewers, in some cases involving locations across private land. Frequently, the relative desirability of such alternate projects can be determined by inspection, particularly when no change in size of sewer is probable; but in other cases it may be necessary to compute the complete design for each project, and make comparative estimates of cost[1] before a decision can be reached. Unless there is a material advantage in cost or other condition resulting from a location through private property, it is generally inadvisable to build sewers outside the public ways.

COMBINED SEWERS OR STORM DRAINS

The design of combined sewers or storm drains requires, first, the determination of the storm run-off as described in Chapter III, and second, the selection of the proper dimensions for the

[1] See Chapter XXI.

sewer, taking into account the available slope and other topographical and physical conditions.

116. Storm Run-off.—The determination of storm run-off or required capacity of the sewer may be arrived at either by the rational method or by the use of an empirical formula. In this discussion, the rational method will be followed. The basic principles of design are the same, whichever method is used.

The computation of the run-off requires the determination of the following basic data:

1. The time-intensity rainfall curve to be used as a basis of design.

2. The probable future condition of the drainage area, *i.e.*, the percentage of impervious surface which may be expected when the district shall have developed to the extent assumed.

3. The run-off coefficient, *i.e.*, the proportion of the rainfall which will run off over the surface of the ground.

4. The probable time required for water to flow over the surface of the ground to the first inlet, called the "inlet time" or "time of entrance."

5. The area tributary to the sewer at the point at which the size is to be determined.

6. The time required for water to flow in the sewer from the first inlet to the above-mentioned point, which added to the inlet time, gives the time of concentration.

Then by the application of the proper run-off coefficient to the rainfall for the time of concentration, the rate of run-off per unit of area may be computed.

117. Adoption of Rainfall Curve.—The form of the rainfall curve will be determined by the records of excessive rainfall for the locality. The frequency of the particular curve to be adopted will depend largely upon economic conditions, and the extent to which it is necessary and financially practicable to minimize surcharging of the sewers and possible damage from flooding. Consideration should be given to the relative position of rainfall curves of different frequencies; thus, if the curve of 15-year frequency lies but little below the 25-year curve, there will be little saving in cost by adopting the former, and it may be better to adopt the 25-year than the 15-year curve as the basis of design. Perhaps a rainfall curve of 15-year frequency is likely to be adopted more often than any other. In many cases, economic considerations may not justify the construction of

drains for flows likely to occur less often than once in 5 years, on the average.

118. Determination of Run-off Coefficients.—An estimate should be made of the coefficient of imperviousness for each sewer district as it is likely to exist at the end of the assumed period of design, say 25 years for instance. Such an estimate can be made with greater certainty where a proper zoning ordinance is in force than elsewhere, although too much credence must not be given to the permanent effect of such an ordinance. Suitable run-off coefficients for impervious and pervious surfaces must then be assumed, bearing in mind the character of the earth and other local conditions which may have an effect. These coefficients may then be combined, either directly or according to the zone principle, and modified if desired, for the effects of distribution of rainfall, retardation, and retention, as discussed in Chapter III, thus obtaining the run-off coefficients applicable to the locality under consideration at the end of the period of design.

119. Economic Considerations.—It must be remembered that run-off coefficients are estimated for future conditions and, consequently, the frequency adopted for design will represent the average interval between occasions when the capacity of the drain will be equalled or exceeded after the district has developed to the assumed condition. During the period of development, the flood conditions would be much more rare, if experienced at all.

This consideration may be illustrated by an example. Assume that a drain is to be built forthwith, to serve an area for which the time of concentration is 45 min. and that the rates of rainfall corresponding to various frequencies and this time of concentration are:

Once in 40 years,	2.30 in. per hour
Once in 20 years,	2.10 in. per hour
Once in 15 years,	2.00 in. per hour
Once in 10 years,	1.85 in. per hour
Once in 5 years,	1.70 in. per hour

Now, if the assumed run-off coefficient when the district has developed be 0.60, and the estimated coefficient at present be 0.40; if the 15-year rainfall curve is to be used; and if it be further assumed that the district develops uniformly, reaching

full development (corresponding to the coefficient 0.60), in 20 years, then:

1. The drain will be designed for a run-off equivalent to $0.60 \times 2.00 = 1.20$ in. in depth per hour.

2. This rate of run-off corresponds to a present rate of rainfall of $1.20/0.40 = 3.00$ in. per hour, which for a 45-min. period, has a probable frequency of about once in 200 to 300 years.

3. The corresponding rate of rainfall required to fill the drain after 10 years, when the coefficient of run-off is 0.50, is 2.40 in. per hour, and the probable frequency of such a rain is once in 70 to 100 years. It is obvious that such a drain is very unlikely to be filled to capacity at any time within 15 years after construction. After 20 years it may be expected that it will be taxed to its capacity or surcharged at average intervals of 15 years.

This being the case, it may be questionable whether the construction of so large a drain is justified. The basis of design should be re-examined and it may be found wiser to design for a condition of development which will be reached at an earlier date, even if still further development goes on thereafter; or a rainfall curve of 10- or 5-year frequency may be adopted. If the basis of design is so modified, it may be expected that at some future time it will become necessary to provide a relief drain, but this may be more economical than to build the larger structure at once.

120. Run-off Curves.—Having determined upon the rainfall curve and run-off coefficients to be used, these may advantageously be combined and a series of run-off curves prepared, similar to those shown in Fig. 27 for Detroit, Mich. From these curves, based upon coefficients combined according to the zone principle, the rate of run-off in cubic feet per second per acre can be taken directly for a given duration of rainfall and percentage of impervious surface.

121. Inlet Time.—As a preliminary to computing the time of concentration, the *inlet time* to be used must be determined. Theoretically, this is the time of concentration at the inlets, or the time required for water to flow from the most distant point to the inlet, and includes the effect of various retarding influences. Many inlets or catch basins now in use are not fitted for the rapid passage of large quantities of water. Allowance must be made, however, for such improvements as are likely to be made, and the adopted inlet time should not neces-

sarily be taken as that required with present gutters and inlets. It is commonly assumed, the limiting values being 5 and 20 min., depending upon local conditions.

122. Data from Maps and Profiles.—The remaining data for the computations must be taken from an accurate plat of the district, similar to Fig. 54. On this are entered the elevations of the proposed or established street and alley grades and, if no contour map has been made, the existing surface elevations should also be shown. The storm-water inlets along the gutters of the streets and alleys are then located on the plat on the higher side of all street intersections, and at all low points between streets, usually with no interval greater than 600 or 700 ft. between inlets. After the situation of the inlets has been fixed on the map, sewers to reach them are laid out, attention being paid to the drainage of all private lots in the district. The most economical layout usually follows the natural surface slopes in the shortest line toward the outlet of the district and concentrates the storm flow as rapidly as possible. Sometimes several preliminary layouts should be made and compared to ascertain which is the cheapest.

The area tributary to the sewer at any point may now be determined. The designer must form a mental picture of the district as it will be with the grading and paving done and buildings erected. This concept is necessary in order to locate the minor ridge lines dividing the small areas draining into streets from those draining into alleys, and to fix the areas tributary to each inlet.

The final step in preparing the data is to fix in a preliminary way the slopes of the sewers. The start in this work of approximating slopes is made at the lower end where the elevation is fixed approximately by outside conditions. Then in the second trial, beginning at the upper end, the final slopes can be established at the same time the sizes are determined.

123. Design of a System of Storm Drains or Combined Sewers.—The following example applies equally well to storm drains and combined sewers. The district shown in Fig. 54 is a portion of the Carlisle Brook drainage area in the City of Springfield, Mass. The location of the proposed main drain (the so-called "Carlisle Brook Drain") into which the district is to be drained, is shown on the plan, and the invert elevation is given at the point where the proposed branch drain is to be connected,

and for which provision has been made in the design of the main drain. The required minimum elevation of the invert of the branch drain is, therefore, given at the proposed point of discharge into the Carlisle Brook drain.

A careful study of local conditions, including the present and probable future development of the district, indicates that a coefficient of imperviousness of 0.40 is reasonable, and that it is proper to assume a coefficient of run-off $c = 0.35$. The zone principle is not applied.

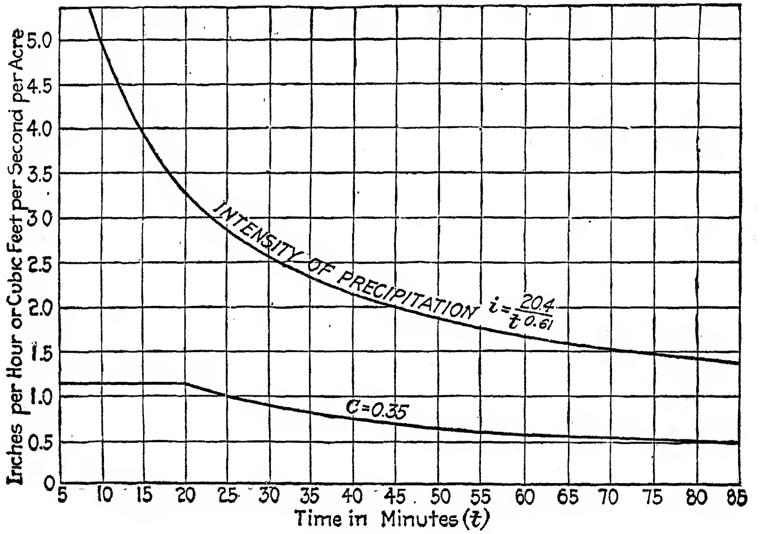


FIG. 55.—Rainfall and run-off curves for Carlisle Brook Drainage District Springfield, Mass.

The inlet time has been assumed to be 20 min.

The rate of rainfall is to be taken from the assumed curve of intensity of precipitation represented by the formula $i = 20.4/t^{0.61}$. This formula indicates the rate of rainfall which may be expected to be equalled or exceeded once in every 5 years. The rainfall and run-off curves are shown on Fig. 55.

While it was recognized that drains designed on this basis might be overtaxed on the average once in 5 years, in view of other financial obligations facing it, the city was not thought to be justified in going to the extent of providing for storms of greater intensity, which would have involved greater cost. During the earlier years of the life of the drains, they will be able to care for

much greater rates of rainfall than later, because the assumed coefficient of run-off is based upon future rather than present conditions. A progressive increase in impervious surface and run-off will be caused by the gradual substitution of roofs and paved areas for unimproved areas. In the future when the district is more densely built up and when the city has available funds, relief drains can be constructed to serve those areas where flooding has become sufficiently serious to warrant the expenditure.

Fig. 54 shows the streets with surface contours, also the drainage area, within the dash-and-dot lines. The limits of this area are influenced not only by the surface contours but also markedly by the areas served by existing combined sewers and drains not shown on the plan.

The drains are to be designed, in general, with the crown at a depth of at least 5 ft. below the surface of the street.

The minimum size of drain is to be 12 in.

The assumed minimum mean velocity is 3 ft. per second when the drain is full.

The capacity of the drains is to be determined by the use of a value of $n = 0.015$ in the Kutter formula for sizes of 24 in. and under, and $n = 0.013$ for sizes over 24 in., it being assumed that the smaller sizes will be of vitrified pipe and the larger sizes of monolithic concrete with smooth interior surfaces.

Procedure.—The following steps must now be taken in order to reach a rational solution:

1. Draw a line to represent the drain in each street or alley to be drained. Place an arrow near each drain to show the direction of flow in it. The drains should, in general, slope with the street surface. It will usually prove to be more economical, however, so to lay out the system that the water will reach the main drain by the most direct route. Also, in general, it will prove best to concentrate the flow from small areas as quickly as possible into one drain.

In some localities the roof water is allowed to discharge upon the ground, thence flowing over the surface to the gutter inlets and the drains. In some such instances, drains are provided only to the last gutter inlet, rather than to a point opposite the last house lot, thereby effecting some saving in cost. This practice is open to the criticism that it does not give equal service to all property and is, therefore, inequitable. In the

example under consideration, it is intended to provide drainage facilities for all property within the district.

2. Locate the manholes tentatively, giving to each an identification number. In this example, a manhole is to be placed at each bend or angle, at all junctions of drains, at all points of change in size or slope and at intermediate points where the distance exceeds 400 ft. Where a good velocity will be available during practically all conditions of flow, and the drain is large enough for a workman to walk without stooping, the interval may be increased to 700 or 800 ft. Sufficient manholes should be built to allow access for inspection and cleaning; later when the profiles are drawn and the final slopes fixed, it may be found desirable to change the location of some manholes in order to have the drains at the most advantageous depth, particularly where the slope of the street surface is not substantially uniform. Other considerations, such as obstacles under the street, may require the installation of additional manholes, due to change in alignment of the drain or special forms of construction involved in junctions or connections with other drains.

3. Sketch the limits of the drainage areas tributary at each manhole. The assumed character of future development and the topography will determine the proper limits.

4. Measure each individual area by planimeter or other methods which will give equally satisfactory results.

5. Prepare a table in which to record the data and steps in the computations of each section of drain between manholes.

The computations for a selected line of this section are shown in Table 42.

Each lateral is then designed in a similar way. If necessary, the design of the submain is modified so as to serve the laterals properly.

124. Low-flow Conditions in Combined Sewers.—Where combined sewers are designed for conditions to be expected many years in the future, studies should be made for flow conditions to be expected during the early years, particularly where the system is provided with relief overflows and the dry-weather flow is collected by an intercepting system and conveyed to some distant point of disposal. Such studies should include estimates of the depth and velocity of flow and a consideration of the operating cost of the sewer resulting from the necessity of cleaning and flushing to remove deposits. In protracted dry spells, flushing of

TABLE 42.—COMPUTATIONS FOR DESIGN OF STORM DRAINS OR COMBINED SEWERS

	From manhole	To manhole	Location, street	Length, ft.	Tributary area: Increment, acres	Tributary area: Total acres	Time of flow: To upper end, min.	Time of flow: In section, min.	Rate of run-off from curve, cu. ft. per second per acre	Total run-off, cu. ft. per second	Diameter, in.	Slope of drain	Velocity, ft. per second	Capacity, cu. ft. per second	Surface elevation at upper end	Fall in ft.	Elevation of invert: Upper end	Elevation of invert: Lower end
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1	1	2	Dawes	300	2.28	2.28	20.0	1.7	1.15	2.6	15	0.0049	3.0	3.7	206.2	1.47	199.85	198.38
2	2	3	Dawes	300	2.40	4.68	21.7	1.7	1.10	5.1	18	0.0037	3.0	5.3	206.5	1.11	198.13	197.02
3	3	4	Dawes	300	2.15	6.83	23.4	1.7	1.05	7.2	22	0.0028	3.0	7.9	206.6	0.84	196.69	195.85
4	4	5	Dawes	165	1.54	8.37	25.1	0.9	1.00	8.4	22	0.0031	3.2	8.4	207.1	0.51	195.85	195.34
5	5	6	Andrew	325	2.16	10.53	26.0	1.6	0.99	10.4	24	0.0029	3.3	10.4	207.4	0.94	195.17	194.23
6	6	7	Burr	400	3.11	13.64	27.6	2.1	0.94	12.8	27	0.0017	3.2	12.8	206.1	0.68	193.98	193.30
7	7	8	Burr	35	6.02	19.66	29.7	0.2	0.90	17.7	30	0.0018	3.6	17.7	203.2	0.06	193.05	192.99
8	8	9	Burr	230	10.14	29.80	29.9	1.1	0.90	26.8	39	0.0011	3.3	27.0	203.2	0.24	192.24	192.00
9	9	10	Burr	240	5.65	35.45	31.0	1.0	0.88	31.2	39	0.0014	3.8	31.3	201.9	0.34	192.00	191.66
10	10	11	Oak Grove Ave.	110	11.91	47.36	32.0	0.5	0.86	40.7	45	0.0011	3.7	40.7	201.6	0.12	191.16	191.04
11	11	12	Oak Grove Ave.	95	11.02	58.38	32.5	0.4	0.85	49.6	48	0.0012	4.0	49.6	202.1	0.11	190.79	190.68
12	12	13	Shattuck	295	5.60	63.98	32.9	1.1	0.84	53.8	48	0.0014	4.4	55.0	202.8	0.41	190.68	190.27
13	13	14	Shattuck	260	19.73	83.71	34.0	1.0	0.82	68.6	54	0.0012	4.4	70.0	203.2	0.31	189.77	189.46
14	14	15	Shattuck	145	11.30	95.01	35.0	0.6	0.81	77.0	57	0.0011	4.3	78.0	203.6	0.16	189.21	189.05
15	15	16	Shattuck	225	2.88	97.89	35.6	0.8	0.80	78.3	57	0.0012	4.5	78.5	203.7	0.26	189.05	188.79
16	16	17	Shattuck	380	13.07	110.96	36.4	1.3	0.79	87.6	57	0.0014	5.0	88.0	202.5	0.54	188.79	188.25
17	17	18	Private Land	165	4.16	115.12	37.7	...	0.77	88.6	57	0.0015	5.0	89.0	196.0	0.24	188.25	188.01

Figures in column 8 are obtained by dividing those in column 4 by 60 and by the figures in column 13.
Figures in column 10 are obtained by multiplying those in column 6 by the figures in column 9.
Figures in column 16 are obtained by multiplying those in column 4 by the figures in column 12.

lateral combined sewers is advisable to remove the solids; flushing of large combined sewers is not likely to be effective and it may be found necessary to provide a special shape of invert, such as a cunette, in order to concentrate the flows and provide better velocities and depths.

Table 43 shows the method of analysis applied to the design in Table 42. For this examination the prevailing rate of dry-weather flow should be used, estimated from the present population and the water consumption, with a reasonable dry-weather rate of ground water infiltration. For the present example, the following assumptions have been made:

1. Present density of population 40 persons per acre.

2. Average rate of flow of domestic sewage 100 g.p.d. per person.

3. Dry-weather infiltration of ground water 500 gal. per day per acre.

The ratio of the average dry-weather flow to the capacity of the the sewer when full shows the proportion of the capacity utilized; the corresponding proportion of depth and of velocity may readily be obtained from a diagram similar to Fig. 38, page 131, but prepared for various values of D and S approximating those appearing in the design. A diagram similar to the above was used in this example, but with a large scale enabling proportions to be read with sufficient accuracy for smaller percentages.

It is found that the velocities in the above example will be very low for the ordinary dry-weather flow, with the likelihood that deposits will be formed. If this sewer were to be part of a combined system provided with relief overflows and with an intercepting sewer for diverting the dry-weather flows to a distant point of disposal, it would be desirable to design the main sewer with a cunette-shaped invert in order that solids would be carried along with the ordinary flows, and not deposited above overflows where they would be likely to be discharged by the first overflow of storm water.

125. Storm Frequency Used in Storm Drain Design.—Local circumstances and conditions, physical and financial, have usually a controlling effect upon the capacity for which drains can be provided to care for extreme maximum rainfalls. The legal responsibility of the community is also an important consideration, although it should not be controlling, since any damage from

TABLE 43.—ANALYSIS OF COMBINED SEWER DESIGN UNDER DRY-WEATHER FLOW CONDITIONS

Number of line	Total tributary area	Average rate of sewage flow				Proportion of full capacity, per cent	Corresponding proportion of		Condition for average dry weather flow	
		Sewage, m.g.d.	Ground water at 500 g.a.d.	Total			Depth per cent of full depth	Velocity per cent of full velocity	Depth, in.	Velocity f.p.s.
				m.g.d.	c.f.s.					
1	2	3	4	5	6	7	8	9	10	11
8	29.8	0.119	0.015	0.134	0.208	0.7	4.5	20.	1.75	0.66
10	47.4	0.190	0.024	0.214	0.332	0.8	5.	22.	2.25	0.81
11	58.4	0.234	0.029	0.263	0.407	0.8	5.	22.	2.4	0.88
13	83.7	0.336	0.042	0.378	0.586	0.8	5.	22.	2.7	0.97
14	95.0	0.380	0.048	0.428	0.663	0.85	5.5	24.	3.1	1.03
16	111.0	0.444	0.055	0.499	0.774	0.9	6.	25.5	3.4	1.28

overflowing must be suffered by members of the community, if not by the entire community as a municipality.

The responsibility of a city for damages of this kind is generally held to depend upon the character of the storm and the courts[1] have held . . . "that rainfalls are differentiated for judicial purposes into ordinary, extraordinary, and unprecedented classes. Ordinary rain storms are those which frequently occur, extraordinary storms are those which may be reasonably anticipated once in a while, and unprecedented storms are those exceeding any of which a reliable record is extant." The usual rule in determining the responsibility of a city was stated many years ago by the New York Court of Appeals, 32 N. Y. 489, as follows: "If the city provides drains and gutters of sufficient size to carry off in safety the ordinary rainfall, or the ordinary flow of surface water, occasioned by the storms which are liable to occur in this climate and country, it is all the law should require."

The question of what constitutes "ordinary" storms still remains. Are storms which may reasonably be expected to occur on an average once in 10 years ordinary or extraordinary? There seems to be no way of satisfactorily answering this question and it will be necessary for the engineer to decide in each case what is the reasonable condition to be met. Legal decisions may aid the judgment, particularly with respect to the legal responsibility of a municipality for flooding due to inadequate storm drains and combined sewers.

Generally, greater damage will result from the insufficient capacity of a main or trunk sewer than from that of a branch drain or lateral; yet the damage from surcharging a large number of branches may be the more serious.

126. Conclusion.—While the problem of determining the quantity of storm water to be carried by drains or combined sewers is still difficult and indeterminate, much advance has been made during recent years in the methods of attacking it. This has been due largely to the securing of accurate records of rainfall, showing duration and intensity. More information showing the actual run-off from rains of known intensity upon areas carefully studied to determine their local characteristics and observations of the time required for the water to reach the sewers, is very much needed, particularly to assist the engineer

[1] Eng. Rec., 1912; **56**, 617.

in making a judicious selection of the coefficient of run-off. There is also need for detailed and long-continued studies of the distribution of rainfall within areas of comparatively small extent, say up to 5 square miles, in order to furnish definite information relating to the area covered by heavy storms, and the rate of diminution of intensity of precipitation as the distance from the center increases. The older empirical formulas have largely given way to rational methods of computation, which enable the engineer to exercise his judgment more readily and design structures peculiarly adapted to local conditions.

Problems

1. Draw a profile for a separate sewer in Dawes, Andrew and Burr Streets, Fig. 52, to discharge into the Oak Grove Avenue sewer at manhole 11. Assume a future population density of 50 persons per acre with a maximum rate of domestic sewage flow of 275 gal. per capita per day and 2,500 gal. per acre per day infiltration. Use other basic data as given on page 170.

2. Assume width of trench = 1.4 times the inside diameter d, + 1 ft., but with a minimum width of 3 ft. for the sewer shown on the profile, Fig. 53, and depth of excavation for pipe sewers computed at 0.2 ft. below grade of invert. Compute the yardage of excavation and the length of pipe of various diameters required.

3. Using the basic data given on page 180, draw a profile and design the storm-water drain for Oak Grove Ave., discharging into the main drain at manhole 10. The ground surface elevations may be interpolated from the contours, Fig. 53.

4. Redesign the Oak Grove Avenue drain of Problem 3, as a combined sewer, using the appropriate data from pp. 170 and 180. Note that the depth of cover for separate sewers should govern.

5. If a U-shaped cunette is to be provided in the 57-inch combined sewer, analyzed in Table 43, what should be the diameter of the semicircular bottom if a velocity of at least 1.5 ft. per second is to be maintained at time of ordinary dry-weather flow? Use $n = 0.013$.

TABLE 44.—RAINFALL AND RUN-OFF DATA USED IN COMBINED SEWER DESIGN BY AMERICAN CITIES

City	Method used	i in. per hour	Coefficient of run-off				Inlet time, min.	Remarks
			Business areas	Downtown residential	Suburban	Rural		
1	2	3	4	5	6	7	8	9
Baltimore..........	Rational	$\frac{105}{t+10}$	0.60–0.70	0.50–0.60	0.40	0.30	4–10	
Boston..............	Rational	$\frac{105}{t+30}$	See remarks				5	10-year frequency; coefficients 0.95–1.00 for roofs, concrete and asphalt pavements 0.70–0.90 for macadam streets and brick sidewalks 0.20–0.70 for earth surfaces. 0.00–0.25 for gravel and sand
Buffalo.............	Rational	$\frac{11}{t^{0.41}}$	0.95	0.60	0.40	....	Varies	
Chicago............	Rational (large areas)	$\frac{28}{t^{0.7}}$	0.90	0.30–0.35	....	0.15	15 usually	Sandy subsoil
	Empirical (small areas)	....	....	....	....	....	....	For small areas $Q = ci^{0.75}A$ $c = 0.8$ to 1.6
Cleveland..........	Rational	Curve from local records	0.60–1.00				7–8	McMath's or Parmley's formulas used as check
New York City								
Manhattan.......	Rational	$\frac{150}{(t+16)}$	0.60				4	
Brooklyn, Bronx, Queens.	Rational	$\frac{120}{(t+20)}$	0.30–0.60				5–10	
Richmond.......	Rational	$\frac{8.91}{t^{0.5}}$	0.65				5–7	

Board of Estimate and Apportionment.	Rational	$\frac{47}{(t+8)^{0.75}}$	for 15-year frequency	0.65				
		$\frac{43}{(t+8)^{0.75}}$	for 10-year frequency					
Philadelphia.........	Rational	$\frac{116}{t+17}$	$c = \frac{2.8p + 20}{1 + 0.03p}$ where p = persons per acre				3	One year frequency
Portland, Ore........	Rational	Local rainfall records		1.00–0.25			5	
Rochester, N. Y.....	Rational	$\frac{12}{t^{0.6}}$		0.90–0.25			5–10 (av. 7)	For small roof-covered areas $Q = 4$ c.f.s. per acre = 4-in. rainfall and 100% run-off.
St. Louis............	Rational	Local rainfall records		0.95–0.10			Observed values 3–5	See table.
San Francisco.......	Rational	Curve		0.85–0.30				
Washington, D. C....	Rational	$\frac{19}{t^{0.5}}$ Also local records		0.85–0.60			5–12	
Cincinnati...........	Rational	$i = \frac{16}{t^{0.5}}$	0.70	0.40	0.30	....	5, 10	15 years
Louisville[1]...........	Rational zone principle of run-off	$i = \frac{33.3}{t^{0.683}}$	0.95	0.30–0.60	0.25	....	10, 15, 20	Clayey soil 15 years.
Schenectady[1]........	Rational zone principle of run-off	$i = \frac{146}{(t + 14.5)}1.055$	0.90	0.40	0.30	....	5, 20	10 years
Detroit[1].............	Rational zone principle of run-off	$i = \frac{131}{t^{.9} + 20.5}$	0.80–0.90	0.50 ±	0.30	....	15	10 years
Springfield[1]..........	Rational zone principle of run-off	$i = \frac{20.4}{t^{0.61}}$	0.90	0.40–0.60	0.30	....	10, 15, 20	Sandy soil 5 years.

[1] Columns (4), (5) and (6) coefficient of imperviousness.

CHAPTER VI

SEWER APPURTENANCES AND SPECIAL STRUCTURES

The most important of the appurtenant structures of a sewer system are the *manholes*, by which access to the sewers is possible for inspection and cleaning. *Drop manholes* and *wellholes* have been developed from ordinary manholes to allow the sewage to fall vertically with a minimum of disturbance to flow. *Flight sewers*, with their inverts arranged like stairways, have also been used for that purpose. Where storm water is taken into combined sewers or drains, *street inlets* are provided, often in connection with *catch basins* intended to retain solid matter washed from the streets, instead of allowing it to be carried into the sewers.

Automatic flush tanks are frequently used to provide a periodic discharge of water in considerable quantity at the dead ends of lateral sewers in which the slopes are flat and there would otherwise be danger of clogging. *Flushing manholes*, operated manually, are sometimes used to retain water or sewage until a considerable quantity has been stored, which can be released suddenly. Occasionally a *flushing inlet* is provided on the bank of a river or pond, for the admission of water for cleaning the sewers.

Where large sewers join there are *bellmouths* and other forms of *junction chambers* to be built, which sometimes assume considerable complexity. *Inverted siphons* are used in crossing valleys or passing beneath subways, streams or other obstacles. A *true siphon* may be used sometimes to overcome a small ridge, although it is considered preferable to go to considerable expense to avoid such a detail. Since reinforced concrete came into use, *sewer bridges* of specially designed hollow girders or beams have been employed in some places to cross rivers or deep gulches, where inverted siphons or steel bridges would have been used before. If a combined sewerage system includes intercepting and relief sewers, some form of *regulating device* is generally required at each place where the sewage is discharged from a

collecting sewer into an intercepting or relief sewer; there are numerous forms of *automatic regulators, storm overflow chambers,* and *leaping weirs* used for such situations.

Where sewage is discharged into a river, lake, or tide water, an *outlet* of some kind is needed. When the discharge is through a submerged outlet into tide water or a stream subject to fluctuation in height, it may be desirable to provide a *tide gate* or *backwater gate* to prevent the water from backing up into the sewers.

INLETS, CATCH BASINS AND MANHOLES

127. Street inlets are provided at intervals along the gutters to admit storm water from the street surfaces to the drains or combined sewers. In general, they are located near street intersections so as to intercept the water before it reaches the crosswalks; and when the distance between cross streets is so great that water might otherwise accumulate in the gutters to a considerable depth, intermediate inlets are provided.

When slopes are steep and gutter velocities correspondingly high, it becomes necessary to depress the gutter at the inlet and raise it just beyond, in order to deflect the water into the inlet. Such depressions and berms are objectionable to vehicular traffic, and accordingly the practicable amount of depression is limited. Unless the inlets are at comparatively short distances apart some of the water may pass such inlets and cause objectionable conditions in the gutters. Sometimes it may be necessary to use elongated or multiple openings in the curb, in addition to gratings in the gutter, in order to admit all the water from the gutter on a steep slope.

Although inlets frequently are located at the angle of intersection of two streets, this is not considered the best practice. At this location they are in the line of heaviest travel which may injure both the inlet casting and adjacent pavement, and moreover this position is unfavorable for the rapid removal of the storm water. The resultant direction of flow of the intersecting streams at this point is away from the inlet rather than toward it. The better arrangement is to place inlets above each crossing used by pedestrians.

There are few data available on the carrying capacity, especially on slopes, of curb openings, which are usually from 2 ft. 6 in. to 4 ft. long and 6 to 8 in. high. Tests to determine the capacity

of inlets and the effect of longitudinal grade in the gutter and of depression of the inlet were made by W. W. Horner of St. Louis.[1] The best type of inlet tested had a clear opening of 6 in. by 4½ ft. The sill sloped abruptly down from the gutter and there was no grating in the gutter. The computed intake capacity of this inlet was 1.0 cu. ft. per second. Fig. 56 shows the measured capacity of the inlet with water just lapping past, for various street grades and depressions in the gutter. For larger discharges in the gutter, the inlet will take greater quantities of water but

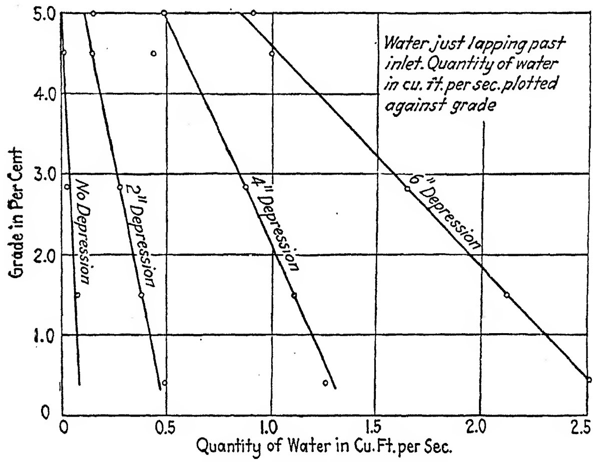


FIG. 56.—Relation between street grade and capacity of inlets, for various depths of depression (St. Louis).
Inlet opening is 4 ft. 6 in. long and 6 in. high.

some will pass on down the gutter. It will be noted that to take 1 cu. ft. per second requires a 4-in. depression on a grade not exceeding 2 per cent, or a 6-in. sump on grades up to 4½ per cent. Horner says that

> . . . the converging lines seem to indicate that with the new style single inlet it would be impossible to take in water on grades of over 7 per cent and that this condition is certainly true for any of the sump depths within the range of this test, and within the range of probability. This indicates, therefore, that *for all steep grades a different type of inlet is required* and an inlet with a long opening . . . seems desirable.

Where there is much waste paper or leaves, a grating placed in the gutter may be of little value in removing storm water, as it may be closed either partially or completely at the first run-off

[1] *Munic. and County Eng.*, 1919; **57**, 147.

from the storm. It would appear advisable in most cases, therefore, to provide an opening in the curb adequate to remove the total flow accumulating at the inlet. Cleanliness of streets and adjacent sidewalks is a great aid to the successful functioning of inlets during storms.

While a perforated cover or grating is frequently used in the gutter at an inlet, some engineers depend entirely upon the curb opening for receiving the storm water and place the cover, if

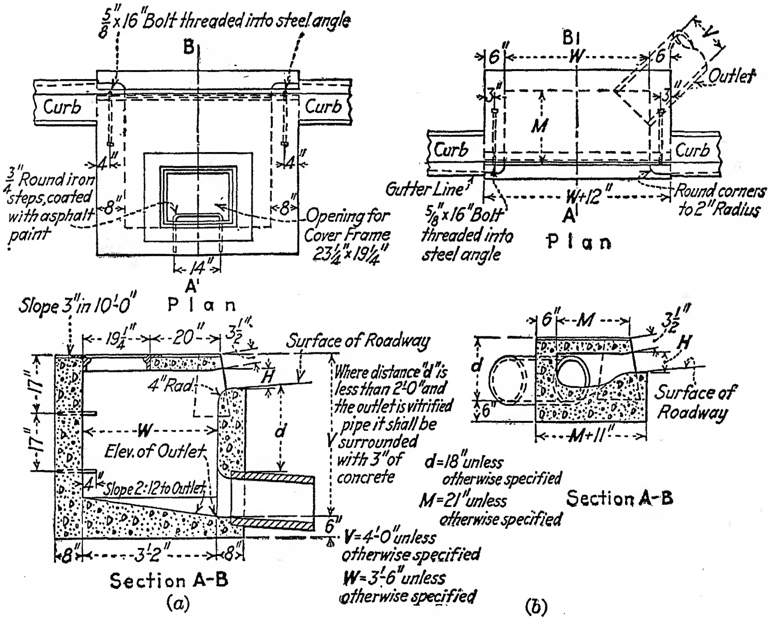


FIG. 57.—Standard inlet, Los Angeles, Calif.

any be required, in the sidewalk. The Los Angeles inlets shown in Fig. 57 are of this type. The greater depth of (*a*) is due to the necessity of carrying the outlet drain beneath the pavement, which is not the case for the shallow inlet (*b*). Since all materials entering these inlets reach the sewer, self-cleansing grades must be used.

In Fig. 58 two inlets are used with a single catch basin, which is sometimes combined with one of the inlets. In this standard of the Borough of Manhattan, the Type *B* inlet is shallow and consists of a casting with a central supporting rib surmounted by an inlet casting and grate of the usual type. The Type *A* inlet

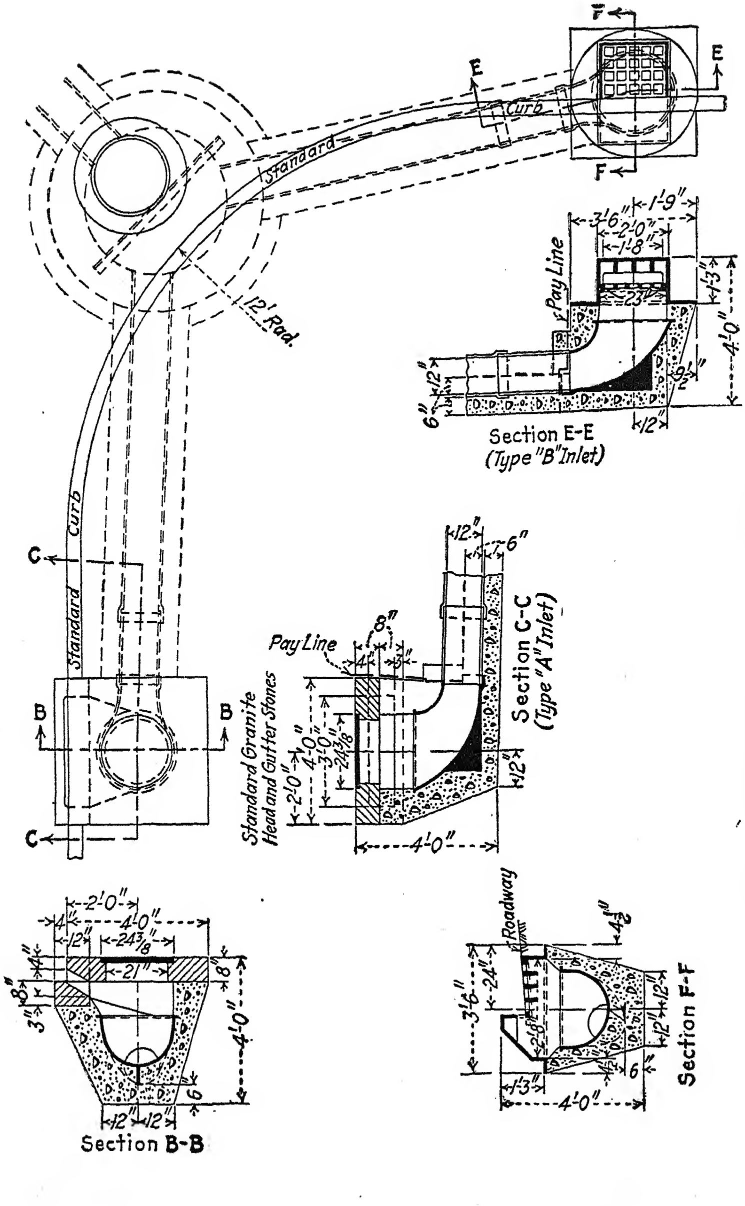


FIG. 58.—Standard inlets, Borough of Manhattan, New York City.

has no opening in the gutter but it can be inspected or cleaned through an opening in the sidewalk closed with a standard basin cover.

128. Catch Basins.—The catch basin was formerly considered an essential part of any American combined sewerage or storm drainage system. Experience had shown that in many sewers the velocity of flow was insufficient to prevent the formation of deposits, and it was manifestly more expensive to remove deposited material from the sewers than from catch basins. This experience was gained in days when the pavements of American streets were crude and little attention was paid to keeping them clean. The sewers themselves were not laid with the present regard for self-cleaning velocities. Under such conditions it was natural that catch basins should find more favor than they do at the present time. Durable pavements, more efficient street cleaning, and sewers laid on self-cleansing slopes have reduced the need for such structures to a few special situations. The following quotation from a report, while made in 1896, still shows the general opinion at the present time.

> We are also of the opinion that the inlets should not be provided with catch basins to retain the filth or whatever may be washed into them. The object of such basins is to intercept heavy matter and periodically cart it away, instead of allowing it to reach the drains and there to deposit. Catch basins, even after the sewage flow no longer exists in the gutters, are still apt to get foul because of the organic matter washed from the street. Such foulness is less offensive in the drains than in the catch basins which are situated at the sidewalk and where it is much more likely to be observed. Also, it is found impracticable to intercept all matter in the catch basins which would deposit in the drains after they reached the flat grade in the lower part of your city. The cleaning of the drains would, therefore, be necessary in any event, and the additional amount of filth that would otherwise be intercepted by the catch basins, will not cost much more to remove.[1]

During recent years the flushing of streets has become increasingly popular. Where this is done, it is useless to have catch basins as they would quickly be filled under the usual conditions of flushing, and thereafter the dirt would be carried into the sewer. Many types of specially designed apparatus are now in use for flushing streets. The horse-drawn flush cart washes more of the refuse into the inlets than do the motor flushers, due to the

[1] HERING, RUDOLPH and SAMUEL M. GRAY, *Report* for Sewerage and Drainage in Baltimore, 1896.

greater quantity of low-velocity water used. With motor flushing, the greater force of the water cuts the refuse from the street surface equally well, while the smaller quantity of water is insufficient to transport all of it to the inlets and leaves much of it in the gutters where it can easily be removed.[1] In some cities, as for example, Rochester, N. Y., catch basins are not used.

The method of cleaning the catch basin may be an important factor in its design. In Chicago, where there were some 130,000 catch basins to be cleaned (1923) and where motor flushing of streets was used, the method of cleaning was by means of a bucket and hoist mounted on a truck. A small orange-peel bucket which can pass through the standard catch basin opening is used in a number of cities. Another effective method of cleaning is by an ejector or eductor mounted on a truck and operated from its engine. The catch basin should be so designed that there shall be no interior arrangements which may be damaged in the cleaning process; and since with a bucket much material is spilled about the opening, a catch basin with opening in the gutter is to be preferred to that opening in the sidewalk.

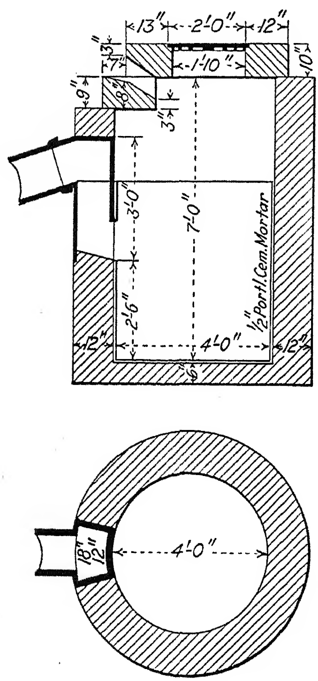


Fig. 59.—Standard catch-basin, Newark, N. J.

The standard Newark catch basin (Fig. 59) is typical of the form in which the trap is in the wall of the basin. The standard catch basin of the Borough of Manhattan is shown in Fig. 60; there are two types, one receiving water only through an opening in the curb, while the other is located under the gutter with storm water admitted both through a curb opening and a grating.

[1] *Am. City*, 1922; 27, 300.

The materials used for catch basins are concrete, brick and stone, the selection for any case depending almost entirely on the cost of the finished structure, since good basins can be constructed of any of them.

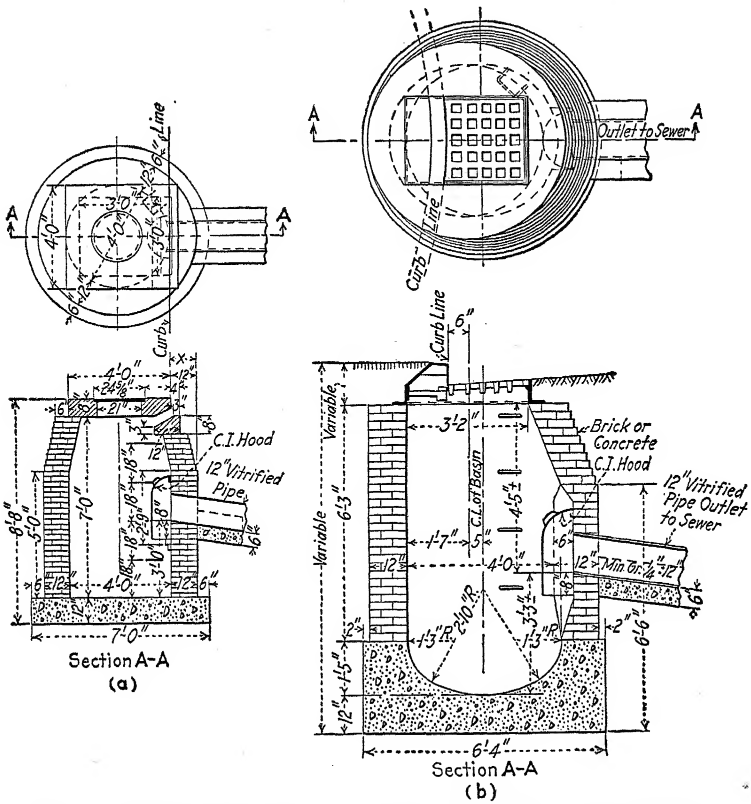


FIG. 60.—Standard catch-basin, Borough of Manhattan, New York City.

129. Castings for Inlets and Catch Basins.—For many years a great variety of castings for catch basins and inlets have been used. The present tendency, however, is toward the adoption of a few standard types designed to meet conditions usually encountered in practice. Fig. 61 shows the standard inlet grating used in Boston, where the preference is for rectangular rather than circular castings. Although ruts are more likely to form in the pavement adjacent to rectangular covers, this shape has the advantage of giving a maximum of grate opening

for a given width of casting, and also has greater stability against tipping. The corners of the support are rounded so that

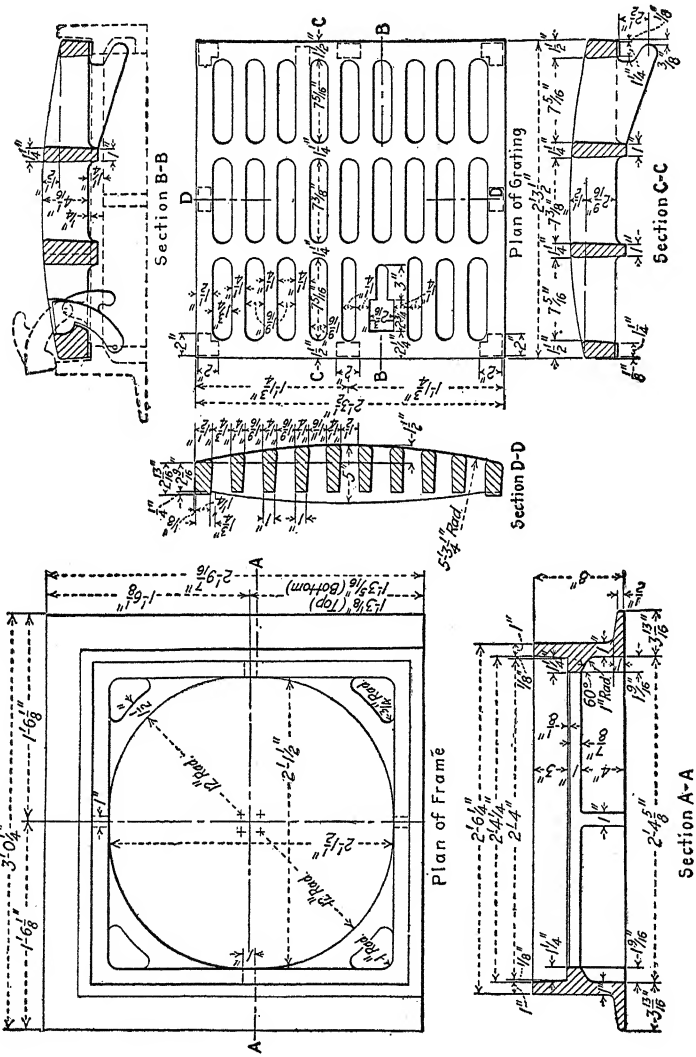


Fig. 61.—Standard inlet castings, Boston, Mass.

the grating cannot be dropped into the basin. The Borough of the Bronx uses the cast-iron inlet head shown in Fig. 62, which has a curb opening as well as the gutter grating.

Whatever type is adopted, it should afford an opportunity for securely bedding the frame upon the masonry of the catch

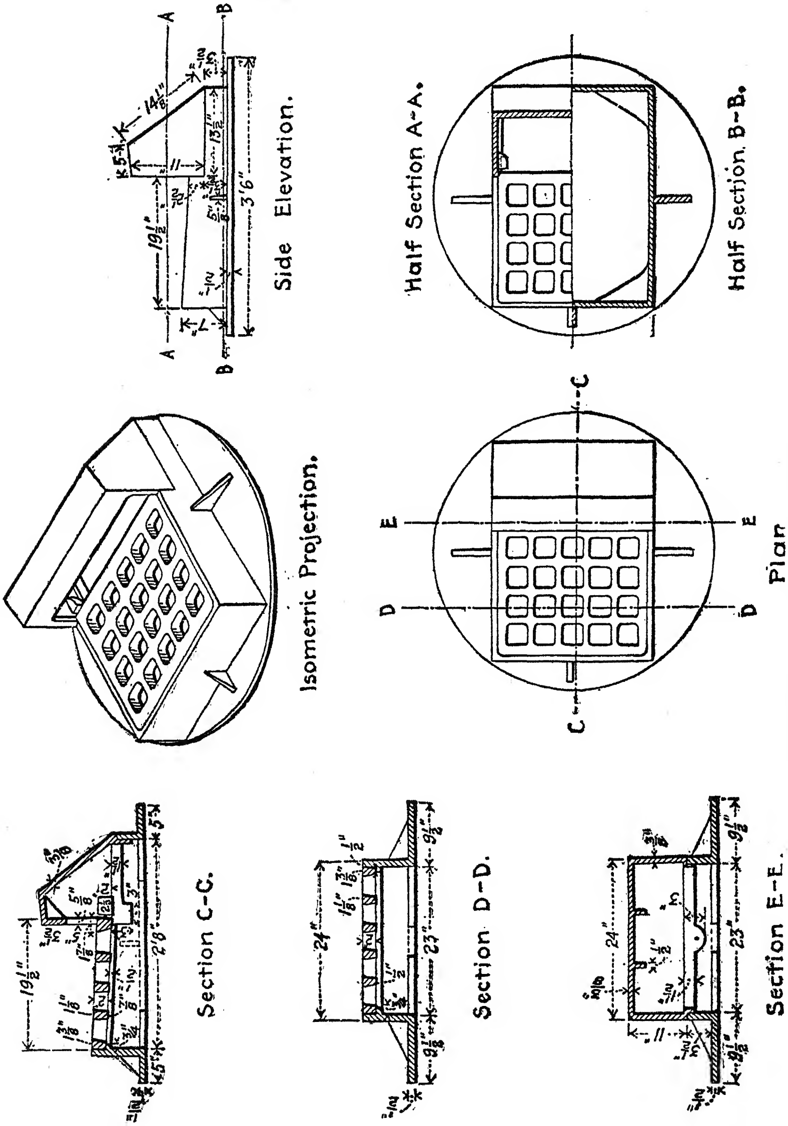


FIG. 62.—Inlet head, Borough of the Bronx.

basin or inlet, for otherwise it will become loosened speedily and in rocking under passing vehicles it will destroy the pavement about it.

130. Manholes.—Although manholes are now among the most familiar features of a sewer system, they were not used extensively on early sewers. Originally they were provided to facilitate the removal of grit and silt which had collected on the inverts of sewers in which the velocity of flow was low. Before manholes were employed, when a sewer became so badly clogged that it had to be cleaned, it was necessary to dig down to it, break through its walls, remove the obstruction and then close the opening, leaving it to cause the same trouble again at a later date. The opposition to manholes seems to have been due mainly to fear of sewer air escaping from them, which fear was not surprising in view of the contemporary accounts of the evil odors from defective drains. Some time elapsed, however, before the value of manholes on small sewers became recognized, and the principle was established that there should be no change of grade or alignment in a sewer between points of access to it, unless the sewer were large enough to enable a man to pass through it readily.

Usually manholes are placed from 300 to 500 ft. apart; but on large sewers, say where the diameter is 48 in. or more, some engineers are using spacings of 1,200 and 1,500 ft., since inspections can be made by entering the sewer itself.

Most manholes are constructed of brick, although under some conditions concrete may be used to advantage. Manholes should be designed so as to provide easy ingress and egress. In some manholes, the clearance opposite the steps is not sufficient for a man to pass up or down without considerable difficulty. On small sewers there should be room to permit of the handling and jointing of the 4-ft. rods used for removing obstructions from and for cleaning the sewer. There should be room to handle a shovel, and the bottom should afford suitable footing for a workman but should drain toward the sewer.

Manholes are usually made about 4 ft. in diameter when of circular cross-section, or about 3 by 4 ft. when an oval cross-section is employed. One size is employed for all sewers except when special conditions may require manholes of larger size, as when gaging devices must be used at the bottom of the manhole, or where it is desired to have considerable storage capacity in the manhole to enable it to be used for flushing. Brick manholes are usually built of 8-in. brickwork down to a depth of 12 to 20 ft. For greater depths 12 in. of brickwork is used as a rule. The sides are generally carried up vertically to within about 5 ft. of

the top, and the upper part is corbelled in or laid in the form of a dome.

Where the sewer is much larger than the diameter of the manhole, the outside of the latter is usually made tangent to one side of the sewer, otherwise it will be difficult to enter the sewer without a special ladder. Occasionally, on very large sewers, the manholes are built entirely apart from the sewer proper and have a passage leading into it, as shown in Fig. 63.

The four manhole bottoms shown in Fig. 64 illustrate somewhat different types of design. The Memphis and Seattle bottoms have flat lower surfaces, while the Concord and Syracuse

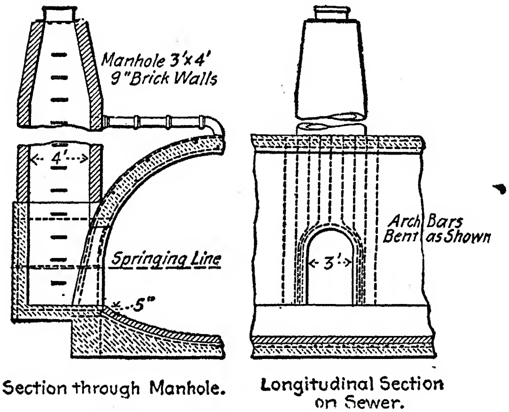


FIG. 63.—Manhole on large St. Louis sewer.

bottoms are curved to correspond with the channels through them. Which type of base is best adapted for the soil at any site can only be ascertained by examination; the saving in material in the second type may be counterbalanced by an increased labor cost. While the base of each manhole illustrated was constructed of concrete, a good sewer mason can readily lay up brickwork to form practically any channel section that may be desired.

The channels in the bottoms of the Memphis and Concord manholes are not provided with high walls, the Concord channel being nearly semicircular and the Memphis channel hardly more than that. The channels of the Seattle and Syracuse manholes have such high walls that they will carry all the sewage until the sewers become surcharged. It is now considered desirable to

have the walls of the channel rise nearly to the crown of the sewer section, and then be stopped in a berm, which is given a slight pitch from the wall toward the channel. The standard Philadelphia manhole bottom, Fig. 65, illustrates the method of giving a little extra velocity to the sewage leaving the branches, by providing a steep grade for the invert within the manhole.

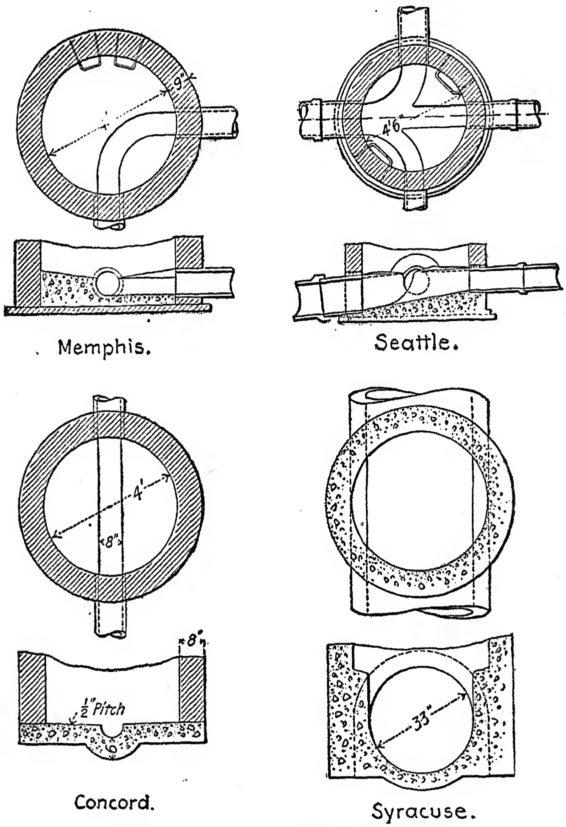


FIG. 64.—Types of Manhole inverts.

Changes in size or shape of cross-section of the sewer as it passes through a manhole produce disturbances in flow with accompanying loss of head. Chamfering corners at inlet and exit, and making all changes of section by gradual transitions, assist in reducing these head losses. Carrying the sidewalls of the sewer up nearly to the crown, as above mentioned, gives greater

uniformity of section at high flows with lower head losses. There is no uniform practice in this detail, however, as the location of the berm at the top of this sidewall varies from a point at mid-depth of the sewer to one level with the crown. Folwell suggests a height of at least two-thirds the sewer diameter; San Francisco practice is to use three-fourths diameter, while the Newark standard carries the sidewall up to the crown elevation. The latter represents the more general present procedure.

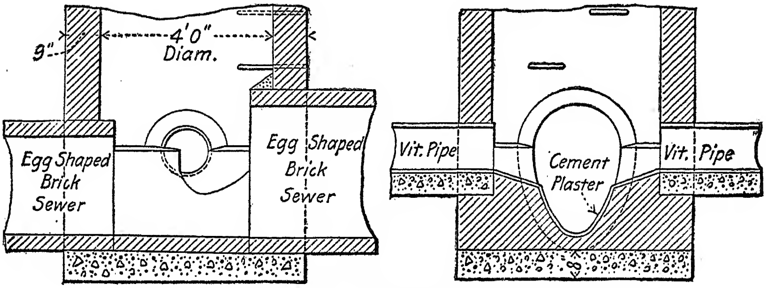


FIG. 65.—Standard manhole invert, Philadelphia, Pa.

131. Drop Manholes.—There are many types of drop manholes, of which one of the simplest is shown in Fig. 66. This particular structure is at the head of an oval sewer 51 in. high, into which two circular sewers discharge at different elevations.

Where sewers, especially small laterals, enter a manhole above the invert, there should be no shelf upon which solids can accumulate and cause offensive odors. Preferably such a sewer should drop outside the manhole and enter at the bottom.

132. Ironwork for Manholes.—*Steps.*—In shallow manholes steps are sometimes formed by leaving projecting bricks at the proper points, about 15 in. apart vertically. This is an old practice and while not generally approved by engineers, was at one time commonly employed. Such steps are objectionable because they are often slippery, when it is difficult to use them safely; they do not provide adequate hand hold; and they are easily broken.

A common method of providing steps at the present time is to construct them of forgings, which are bedded in the brickwork or concrete. Three types of such steps are shown in Fig. 67. The steps are usually placed from 12 to 18 in. apart vertically

and staggered; a number of cities seem to be in favor of a vertical spacing of 15 in.

On some work which has come to the authors' attention, it has been found that the material used for making forgings for

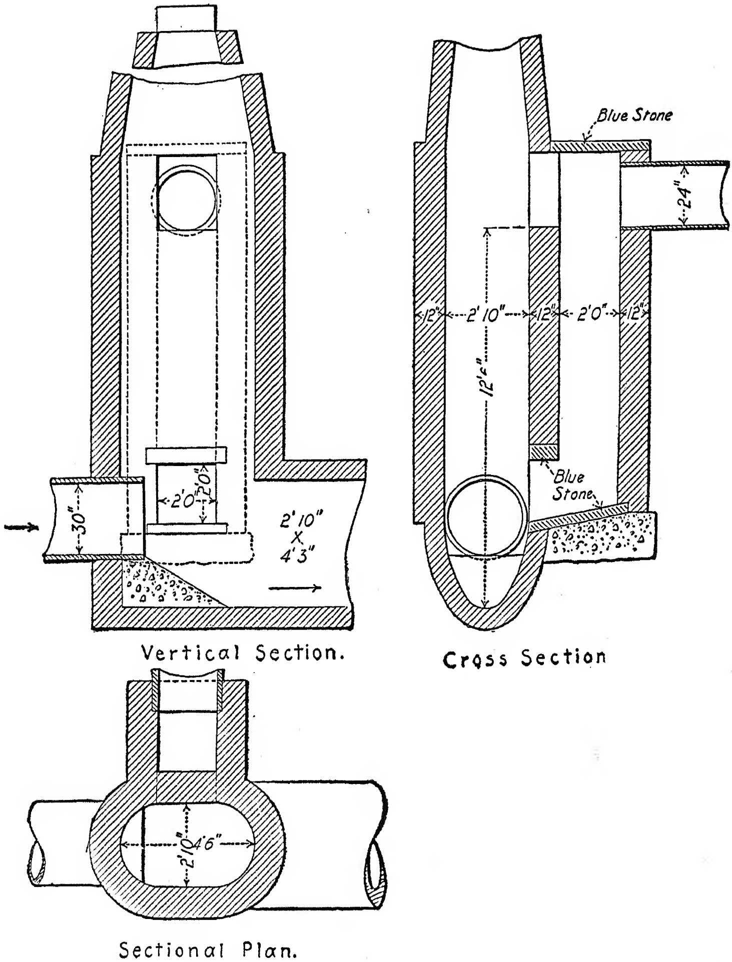


FIG. 66.—Drop manhole, Newark, N. J.

steps has rusted to such an extent that the steps are practically useless. In one instance this condition was found where genuine wrought-iron steps had been specified. Present practice (1929) in several cities, is to use cast-iron steps as shown in Fig. 68.

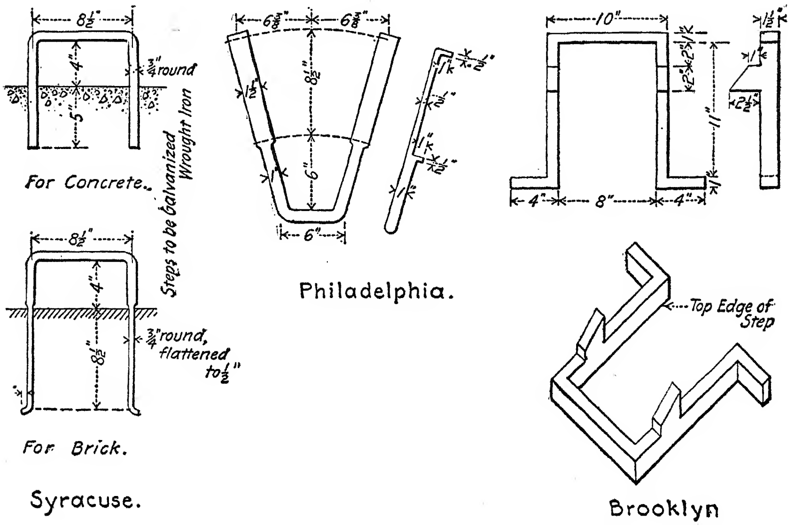


FIG. 67.—Types of step forgings for manholes.

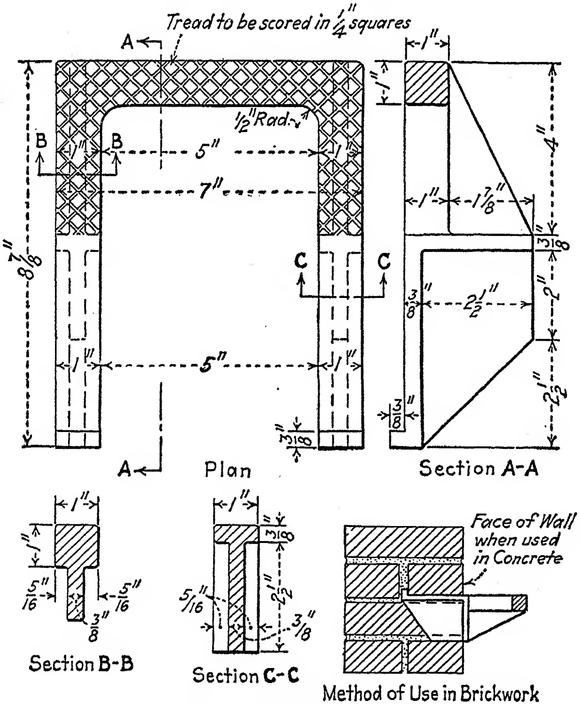


FIG. 68.—Cast-iron manhole step.

Manhole Heads.—It is only within the last few years that the design of manhole heads has been given serious attention. The objects to be attained in the design of castings particularly for manholes, are: safety, so that covers will not slip off; convenience of repair and replacement, necessitated by the wear from traffic; strength sufficient to stand up under increasing wheel loads; freedom from rattle and noise; low cost; possibility of adjustment with the wearing down of pavements to prevent unevenness with its accompanying inconvenience to traffic and increased wear on the pavement; appearance; ventilation to remove accumulating gases so as to increase safety of workmen in manholes and sewers; protection against ingress of water, (especially for manholes used by telephone or electric light companies); protection against admission of lighted cigars and cigarettes; and protection by locking devices against removal of covers for dumping of refuse into the opening.

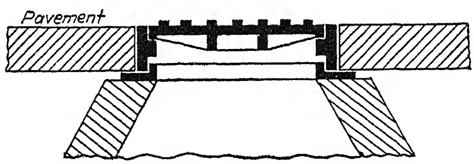


Fig. 69.—Reversible manhole frame, Chicago.

Covers should be interchangeable for convenience in replacing those lost from theft or breakage. In the borough of Manhattan, with more than 30,000 sewer manholes, about 1½ per cent of such replacements are required each year. The cover should be corrugated or provided with bosses to prevent slipping. Circular tops are in almost universal use for sewer manholes. They are inherently stronger than rectangular ones, and have the advantage that it is impossible to drop the cover into the manhole.

Frames are usually from 6 to 12 in. in height, depending partly upon the type of pavement in which they are to be installed. Practice as to clear opening varies widely; a 24-in. cover, allowing 22-in. clear opening, is generally satisfactory. In general, frames weigh from 250 to 500 lb., and covers from 100 to 150 lb.

Manhole frames and covers are usually of cast iron, but semisteel or cast steel may be used. T. J. Corwin[1] describes tests of large manhole covers (28 to 36 in.) made for the Pacific Gas and Electric Company, which indicate that semisteel is the most

[1] *Trans.*, Am. Soc. C. E., 1927; **91**, 991.

advantageous material for large covers. Several designs of cover were tested.

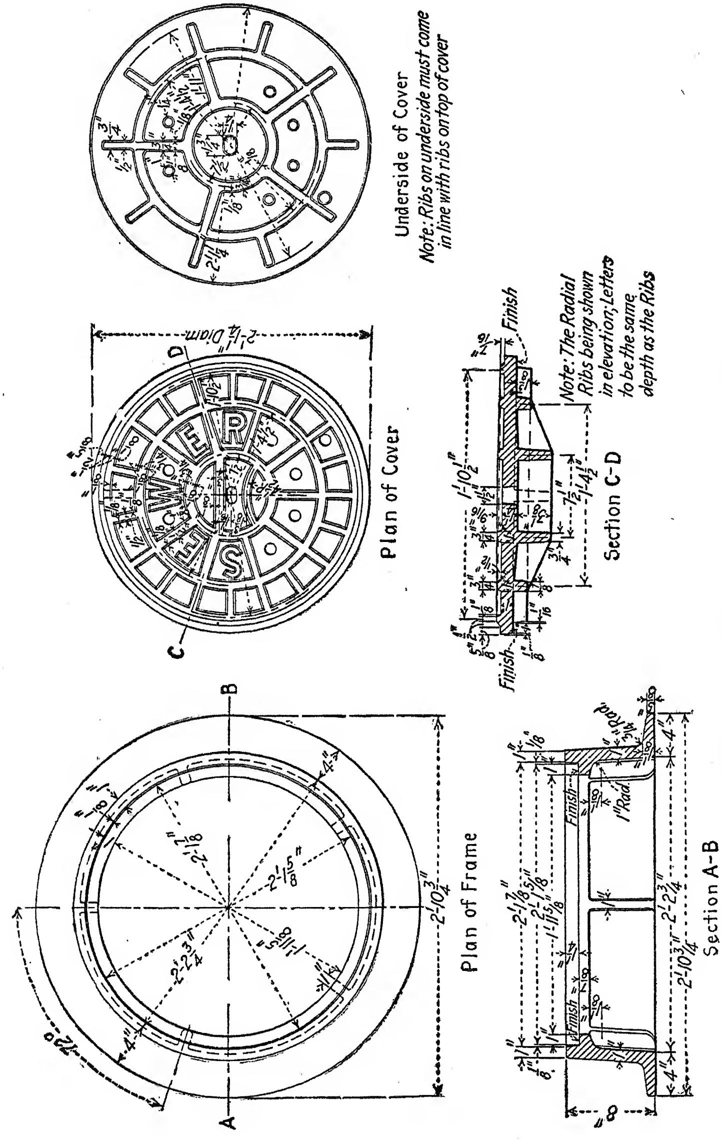


FIG. 70.—Standard manhole head, Boston, Mass.

Wear on manhole frames in large cities results in considerable expense for renewals, the cost of labor and of cutting and replac-

ing pavement around the frame generally being of the most consequence. To reduce this cost to a minimum, the Chicago Sewer Department has designed a manhole head consisting of three pieces, base, curb and cover, as shown in Fig. 69. The base is permanent and remains in place during repairs. The curb is a reversible ring with vertical sides so that it can be removed without damage to the surrounding pavement when worn sufficiently to require reversal. When the reverse edge is also worn down, the curb can be replaced at comparatively small cost. The life of the head is thus prolonged and such replacements as are necessary are simple and inexpensive, causing only slight interruption of traffic.

The Boston standard manhole head, Fig. 70, may serve to illustrate the usual present (1929) practice. The cover is heavier than the older standards and the deeper ribs on the lower side make it more difficult for boys to remove, since it must be lifted vertically several inches. It is perforated, but the number of holes is not large. The clear opening in the frame is 24 in. in diameter.

The quantity of storm water entering through the perforations in covers is difficult to estimate, because of the leakage into the sewers from other sources. As nearly as could be ascertained 3,000,000 gal. of water in 24 hours entered a system including about 125 miles of sewers, through 1,965 manholes having perforated covers, or an average of 1.1 gal. per minute per manhole, during a heavy storm.

So much storm water has found its way into separate sewers in some places that there is now a tendency to use tight covers or those having few and small openings, in an effort to prevent or reduce leakage through manhole covers.

JUNCTION CHAMBERS

Where large sewers come together with a horizontal angle between their axes less than about 30 deg., a special structure called a *junction chamber* is required. For many years these junction chambers were of the type shown in Fig. 71, and were sometimes called "bell-mouths" or "trumpet arches" on account of their shape. Where, at the springing lines, the outside surfaces of the arches of the two sewers come together, no attempt is made to have the upper part of the two sewers independent,

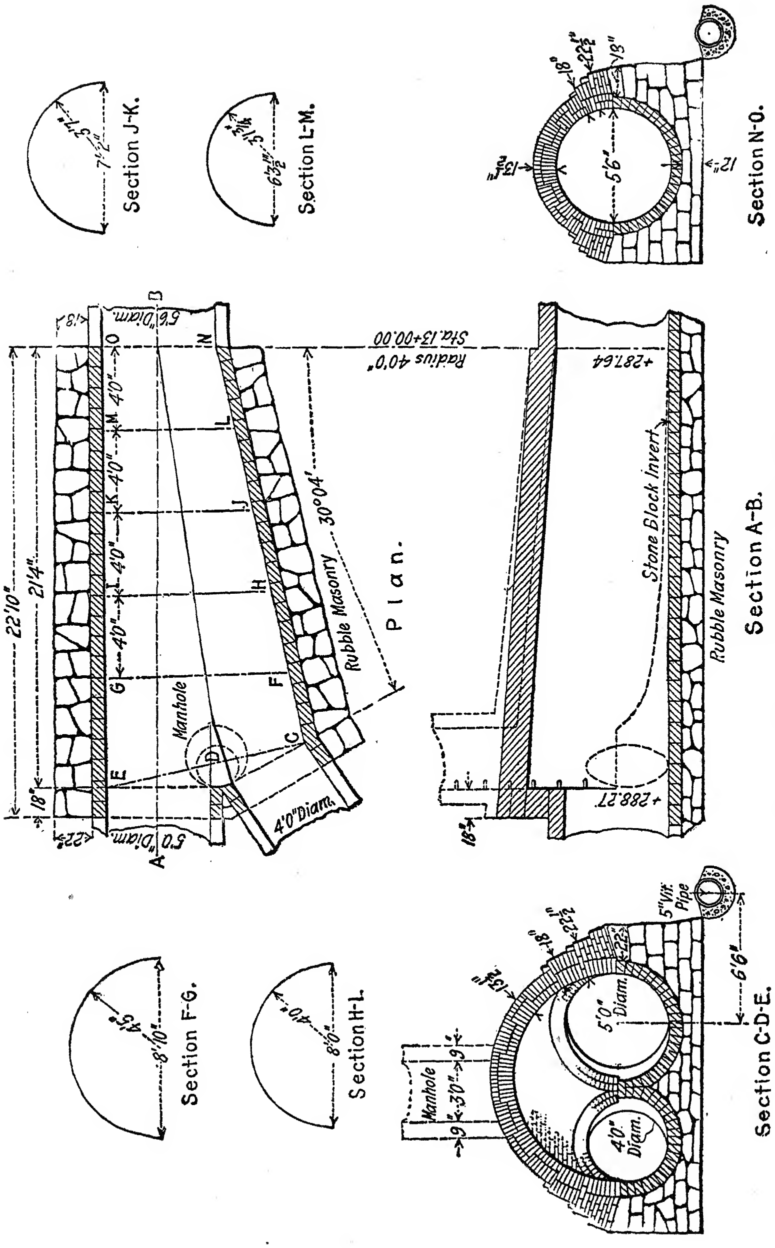


FIG. 71.—Bellmouth junction, Philadelphia.

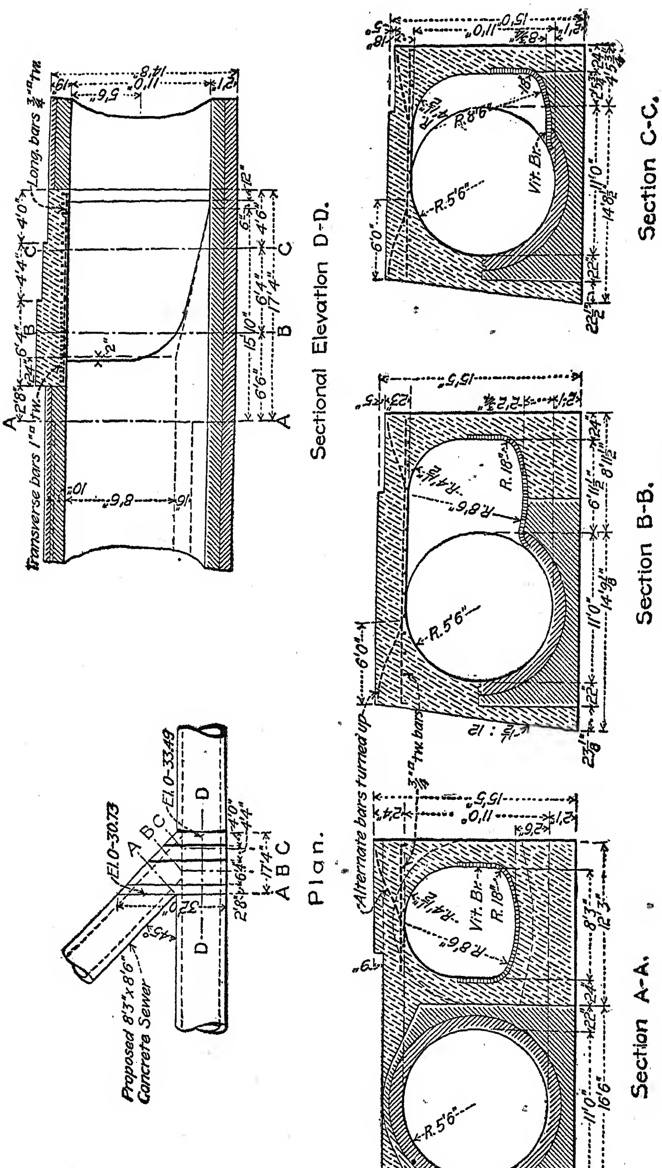


FIG. 72.—Flat-top junction, Pittsburgh.

but a large arch is thrown across the two. A manhole is usually built just in front of the wall which closes the large end of the structure.

These bell mouths were formerly built of brickwork, but concrete is now generally used. So much skilled labor is required in constructing them of either material that flat-topped junction chambers, Fig. 72, have come into use. The particular junction chamber illustrated is rather more expensive than most of this type, because it was constructed on a large existing brick sewer which it was desired to disturb as little as possible. The hydraulic conditions resulting from introducing a large branch into an existing sewer are not ideal, since the greater depth below the junction must result in backing up and a condition of non-uniform flow for some distance above; accordingly this example is significant only for its structural design.

133. Design of Junction Chambers.—In designing junction chambers it is desirable to have the streams of sewage join in such a way that there shall be a minimum checking of the current in either sewer, which might cause deposit of grit and other suspended matter. This condition requires the streams of sewage to have the same surface elevation as they approach each other, and the same velocity, which is not easy to bring about when one sewer is large and the other much smaller. If the invert of the smaller sewer is placed at an elevation which will make the surface elevation of both streams identical during dry weather, the two streams will probably have materially different elevations when considerable storm water is flowing. Consequently a study of conditions at the junction when the two sewers carry various quantities of sewage may show that, to prevent sewage being backed up in the smaller sewer during periods of large discharge, it will be desirable to increase the grade of the smaller sewer somewhat for a short distance from the junction.

Velocities of flow in sewers usually are not high enough so that it is necessary to give special consideration to the hydraulic features of transition structures at junctions. If reasonable care is taken to avoid sudden enlargements and contractions and to make changes of direction by smooth and reasonably flat curves, no difficulty is likely to develop. The condition is radically different, however, when high velocities are involved, say, over 6 or 8 ft. per second. In such cases, the theoretical

position of the flow line from point to point should be computed (by the application of Bernouilli's theorem) and the design of the structure should be changed, if necessary, to obtain a smooth curve for this line, tangent to the water surfaces at each end of the transition.[1]

INVERTED SIPHONS

132. General Features of Inverted Siphons.—Any dip or sag introduced into a sewer to pass under structures encountered, such as conduits or subways, or under a stream or across a valley, is termed an *inverted siphon.* It is misnamed, for it is not a siphon, and the term *depressed sewer* has been suggested as more appropriate. Since the pipe constituting the inverted siphon is depressed below the hydraulic grade line, it is always full of water under pressure although there may be little or no flow in the sewer.

135. Size and Velocity.—Practical considerations, such as the increased danger of stoppage in small pipes, tend to fix the minimum diameters for inverted siphons about as for ordinary sewers, 6 or 8 in. in separate systems and about 12 in. in combined systems. As obstructions are much more difficult to remove from an inverted siphon than from a sewer, especial care should be taken to prevent their formation. As high a velocity as practicable should be maintained in the inverted siphon, say 3 ft. or more per second for domestic sewage and 4 to 5 ft. per second for storm flows. In some cases catch basins or grit chambers have been built just above the inverted siphons, but these are troublesome to clean and the material removed from them is usually offensive. Inverted siphons should be flushed frequently and their operation inspected regularly, in order to assure prompt removal of incipient obstructions.

Experience has shown that the smaller pipes are sometimes clogged as a result of sticks catching in the bends, these sticks presumably having been dropped by children through perforations in manhole covers. When such conditions have been experienced or are likely to occur, it would be advantageous to provide for racks in front of the inlets to the smallest pipes, preferably so arranged that the material collected on them may be washed off and carried through the larger pipes at times of high flow.

[1] Valuable suggestions will be found in a paper by HINDS, JULIAN: The Hydraulic Design of Flume and Siphon Transitions. *Trans.*, Am. Soc. C. E., 1928; **92**, 1423.

136. Means for Flushing.—Flushing or cleaning may be accomplished in various ways, depending upon the available facilities and surrounding conditions. It may be done by receiving the sewage at the head of the inverted siphon in flush tanks which automatically discharge when full; or by providing a permanent flushing gate in the manhole at the head of the inverted siphon; or, if permissible, by opening a blow-off at the low point of the pressure pipe; or by admitting clean water at the head of the inverted siphon from a permanent connection with an accessible stream, a water main or a neighboring hydrant; or by cleaning by hand by the use of jointed rods with suitable scrapers or other tools, after draining the inverted siphons.

137. Manholes or Clean-out Chambers.—Manholes or clean-out chambers should be provided at each end of an inverted siphon, to give access for rodding, pumping, and in the case of pipes of larger size, for entrance. There is objection to the introduction of intermediate manholes on an inverted siphon in such a manner that the sewage will be free to rise in them, since grease and other scum tends to fill the shaft. They are advantageous, however, if the sewage be confined within the pipes as it passes through the manhole, affording access or means of ridding the siphon of deposit, through a gated connection or similar device.

138. Materials of Construction.—Since an inverted siphon is subjected at all points of its cross-section to an internal pressure the walls will be in tension, although the magnitude of the tension will be affected by external water and soil pressures. On account of these tensile stresses, inverted siphons are usually constructed of steel, cast iron, or reinforced concrete.

139. Size of Pipe.—The computation of the sizes of pipe for inverted siphons is made in the same way as that for sewers and water mains. The diameter depends upon the hydraulic slope and the maximum quantity of water to be carried. The head or drop in the hydraulic gradient actually required at any time, for the existing flow, will be the difference in level in the free water surfaces at the two ends of the siphon. It will equal the sum of the friction head and other losses. It is to be remembered that the losses are relatively small for low velocities, other things being equal, but that they increase roughly as the square of the velocity. For a clean 12-in. siphon 50 ft. long and a velocity of 3 ft. per second, a total loss of 6 in. would probably be an outside figure; for a velocity of 2 ft. per second, 3 in.; but

for 6 ft. per second, 24 in.; the friction loss alone being not over one-third of the above values. Experience in this country has shown the advantage of using several pipes, instead of one pipe, for the inverted siphon, arranged in such manner as to throw additional pipes progressively into action with increase in flow of the sewage. In this way reasonable velocities are maintained at all times.

Care must be taken that inverted siphons built on or under river beds have sufficient weight or anchorage to prevent their flotation when empty.

140. Solution of an Inverted Siphon Problem.—A problem encountered in the authors' practice involved the design of an inverted siphon of several pipes, to replace an existing single-pipe siphon which had given trouble from sedimentation due to low velocities; its solution is given as an illustration. The basic data are as follows:

Length of inverted siphon	440 ft.
Available fall (invert to invert)	3.2 ft.
Maximum depression of siphon	9 ft.
Gravity sewers connected by inverted siphon:	
Diameter	30 in.
Material	Concrete
Slope	0.0037
Typical rates of flow	
Minimum	4 cu. ft. per second
Maximum dry weather	13 cu. ft. per second
Ultimate maximum	capacity of gravity sewer

The capacity of the 30-in. concrete gravity sewer on slope 0.0037 (n = 0.013) is about 25 cu. ft. per second, and the velocity of flow 5.1 ft. per second.

Available fall	3.2 ft.
Assumed loss at inlet	0.4 (see later discussion)
Available for friction in inverted siphon	2.8 ft.

$\frac{2.8}{440}$ = 0.0064, approximate available hydraulic slope.

Local conditions indicate economy in the use of vitrified pipe. The surrounding concrete (primarily for anchorage) can be easily reinforced for the internal pressure due to the 9-ft. depression of the siphon. Standard commercial vitrified-pipe sizes will therefore be selected.[1]

Two conditions must, if possible, be satisfied; the velocity in the pipes selected should be sufficient to insure scouring (3 ft. per second, if possible); and the available hydraulic slope of 0.0064 must not be exceeded.

[1] Note that 20- and 22-in. sizes, used in this example, are no longer standard; 21-in. was not standard until recently.

Judgment suggests the selection of pipe sizes which will be particularly adapted to the minimum flow, the maximum dry weather flow and the ultimate maximum flow. Three siphon pipes are found to meet these requirements, with regulation at the inlet so that the minimum flow will be confined to one pipe; the maximum dry-weather flow to two pipes; and all three will have a capacity equivalent to that of the gravity sewer in which the inverted siphon is to be inserted. The computations follow:

For minimum flow, 4 cu. ft. per second; from Fig. 32.

(n = 0.015 for vitrified pipe) 15-in. pipe requires s = 0.006; velocity, 3.3 ft. per second.

Capacity at S = 0.0064, 4.2 cu. ft. per second (beyond which second pipe begins to operate).

For maximum dry-weather flow of	13.0 cu. ft. per second
Capacity of 15-in. pipe	4.2
Required capacity of second pipe	8.8 cu. ft. per second
From Fig. 32, 20-in. pipe, S = 0.0057; velocity, 4.1 ft. per second.	
Capacity of 20-in. pipe at S = 0.0064	9.3 cu. ft. per second
Capacity of 15-in. pipe	4.2
Combined 15- and 20-in	13.5 cu. ft. per second (beyond which third pipe begins to operate).
For ultimate maximum	
Capacity of 30-in. sewer	25.0 cu. ft. per second
Combined capacity, 15- and 20-in. siphons.	13.5
Required capacity of third pipe	11.5 cu. ft. per second
From Fig. 33 22-in. pipe requires S = 0.0058, velocity 4.3 ft. per second.	
Capacity of 22-in. pipe at S = 0.0064	12.0 cu. ft. per second
Combined capacity of 15- and 20-in. pipes . .	13.5
Total available capacity	25.5 cu. ft. per second (25 required).

Design of Inlet Chamber.— A type of inlet chamber, Fig. 73, suitable in this instance, has the invert of the pipe which carries the low flows continuous with that of the 30-in. gravity sewer; one side of the sewer is cut down to an elevation which permits overflow to the second siphon when the capacity of the first is exceeded; and the other side of the sewer is cut down to a higher elevation, which allows the flow in excess of the capacity of the first two pipes to reach the third.

Computations of the depth of flow in the 30-in. gravity sewer when the 15-in. siphon pipe shall have reached the limit of its capacity, (4.3 cu. ft. per second), and when the combined capacity of the 15- and 20-in. siphon pipes (13.5 cu. ft. per second) is flowing, determine the elevations of the tops of the walls, which permit overflow to the 20- and 22-in. pipes, to be

about 8 and 16 in. respectively. They are subject to some error owing to the uncertainty of the formula when applied to circular conduits partly filled.

Under maximum conditions for either of the two larger pipes, the overflow walls will be submerged both upstream and down. They cannot, therefore, be considered as weirs, but as obstructions causing certain losses of head in passing the desired quantities of sewage. The flow over these walls is at right angles to that in the approaching sewer,[1] so that this loss may be taken as the head required to produce the necessary velocity across the top of the wall, assuming the energy of velocity of approach to be lost in the change of direction.

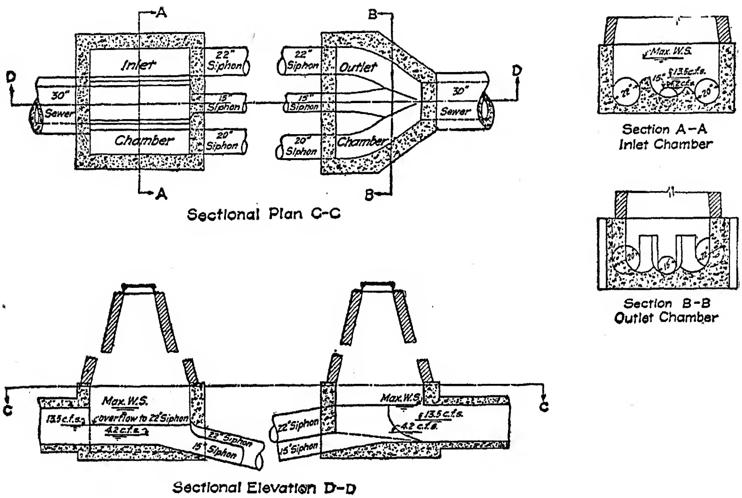


FIG. 73.—Inverted siphon inlet and outlet chambers (problem in inverted siphon design).

A maximum of 9.3 cu. ft. per second must pass across the wall to the 20-in. pipe. The depth on the wall will then be at least 8 in. or the difference in elevation between the two walls, the higher wall having been figured to overflow just as the capacity of the first two pipes was exceeded.

The higher wall must pass at least 11.5 cu. ft. per second, and the depth available is the distance from the top of the wall to the crown of the 30-in. sewer, or 14 in.

Assume the length of walls at 7 ft.; the inlet losses may be approximated as follows:

In 15-in. pipe; negligible, since invert is continuous with that of 30-in. sewer, and transition can be made to take advantage of the velocity of approach which, even at low flows in the gravity sewer, is equal to or greater than the siphon velocity. (Velocity of approach may be estimated by the method given in Chapter IV for the partially filled sewer.)

In 20-in. pipe; velocity over wall, $9.3/(0.67 \times 7) = 2.0$ ft. per second; corresponding head, 0.06 ft. After passing over wall, velocity of 4.3 ft.

[1] Assumption is not strictly in accordance with facts but provides a basis for computations which are on the safe side.

per second in 20-in. pipe must be built up in new direction,[1] which gives a loss of head of 0.29 ft. Total 0.35 ft. (less than assumed inlet allowance of 0.4).

In 22-in. pipe; velocity over wall $11.5/(1.17 \times 7) = 1.4$; corresponding head, 0.03 ft. Velocity in 22-in. pipe, 4.3 ft. per second; head, 0.29; total, 0.32 ft. Allowance of 0.4 ft. for inlet losses appears ample, and no revision of the computation of available slope is required.

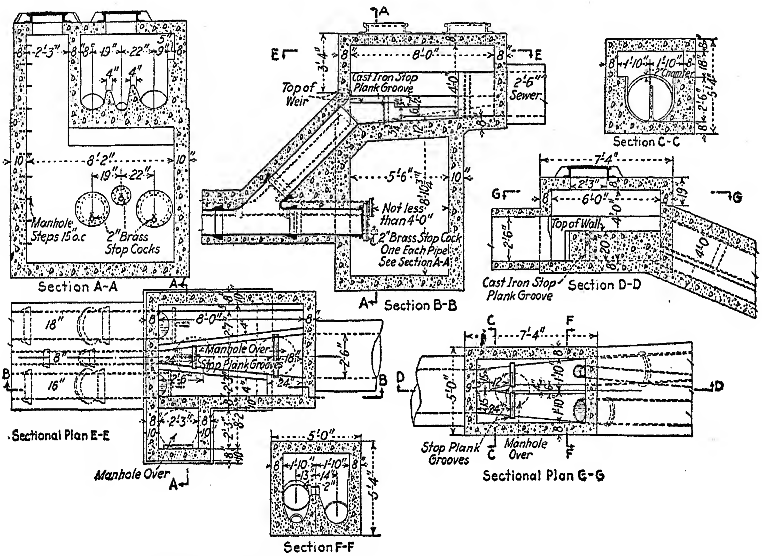


FIG. 74.—Inverted siphon on Middle Fork trunk sewer, Louisville, Ky.

The relative elevations of the pipe inverts are now readily found. The capacity flow of the 15-in. pipe requires a depth of 8 in. in the 30-in. gravity sewer. Since there is no velocity head loss to be allowed for, the crown of the 15-in. sewer is 8 in. above the 30-in. invert. The crown of the 20-in. pipe will be at an elevation of 16 in. minus the entrance head loss of 0.35 ft. ($4\frac{1}{4}$ in.) or $11\frac{3}{4}$ in. above the 30-in. invert. The crown of the 22-in. pipe is 0.32 ft. (4 in.) below the 30-in. crown. From these figures, the relative invert elevations may be determined.

Design of Outlet Chamber.—At the lower end of the inverted siphon, the junction of the three pipes with the 30-in. gravity sewer should be so designed as to reduce the opportunity for eddies to carry sediment back into those pipes which are not at the moment operating, but which are full of standing water. This is especially important in the case of the 22-in. pipe, which will not be in operation except at unusual rates of flow. It may be accomplished by maintaining the three pipes, or the corresponding channels within the junction chamber, as nearly up to the point of intersection as possible, to avoid pooling and reduction of velocity in the chamber. As a further precaution, the outlet of the 22-in. pipe (least frequently required)

[1] Assumption is not strictly in accordance with facts but provides a basis for computations which are on the safe side.

may be raised so that the invert of its channel has a sharp forward pitch toward the intersection. The crown of the pipe must not, however, be raised above that of the 30-in. sewer, or it will lie above the hydraulic gradient.

This solution has also been used in the outlet chamber of the inverted siphon shown in Fig. 74. Here the 18-in. pipe, which is the last to come into service, and which will be full only when the 30-in. sewer into which it discharges is also full, is elevated till its crown is continuous with that of the 30-in. sewer.

141. Insertion of Inverted Siphons in Old Sewers.—It sometimes becomes necessary to insert an inverted siphon in an

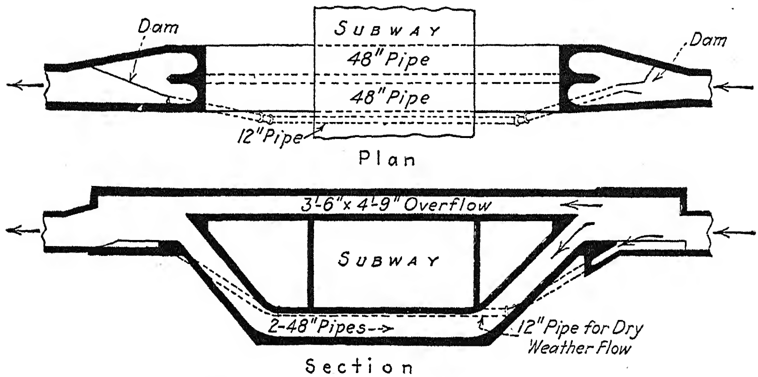


FIG. 75.—Inverted siphon under subway at Mortense St. and Nostrand Ave., Brooklyn, N. Y.

existing sewer when some underground structure is to be built across its course. Usually a readjustment of the sewer gradient is not feasible, and the additional head required must be derived from a raised upstream water surface. This increases the tendency toward deposition of solids in the sewer thus affected by backwater, which may necessitate frequent inspections and flushing. The construction of subways has required the employment of many such inverted siphons which have been operated successfully. The first structure of this type built in connection with the New York subways was at 149th St. and Railroad Avenue, and was placed in service in February, 1902. An example of this type of structure is seen in Fig. 75, which is taken from a report[1] on "Inverted Siphons for Sewers." This siphon is on a combined sewer and provides a 12-in. cast-iron

[1] *Jour.*, Boston Soc. C. E., 1921; **8**, 237; also *Munic. Engrs. Jour.*, March, 1917.

pipe for dry-weather flow and 48-in. circular concrete pipes for ordinary storm flows, while the excess at times of extreme flow is carried over the top of the subway by an overflow channel measuring 3 ft. 6 in. by 4 ft. 9 in.

FLUSHING DEVICES

The primary object of flushing devices is to permit sewers to be laid on flat slopes which may not be sufficient to produce scouring velocities at all times. The problem of flushing is principally that of keeping small sewers clean, from their dead ends to the points where the flow of sewage is great enough to accomplish this without supplying additional water. Occasionally the problem is one of furnishing a large volume of water to clean a main sewer or an inverted siphon. The quantity of water used in one flush is rarely over 350 gal.

142. Flushing from Brooks.—In some places where a sewer is laid near a brook or river, flushing chambers may be built with a connection to the stream through which the sewer can be flushed by admitting brook water.

143. Flushing manholes are merely ordinary manholes which are filled with water or sewage, while the outlet is closed by a valve or plug. When the plug is withdrawn the accumulated water is discharged into the sewer. They may be made somewhat larger than usual if it is desired to make the quantity of flushing water discharged greater than can be stored in the standard manhole; or they may be fitted with permanent valves to avoid the necessity of using plugs, which rarely fit well.

144. Automatic Flush Tanks.—The flushing done with automatic apparatus is generally more frequent than where hand flushing is practiced, the usual rule being to empty the flush tank each day. The water ordinarily enters the tanks through special orifices, of which a variety are manufactured by the makers of automatic siphons, so that any desired rate of flow under any street main pressure can be obtained by screwing the proper orifice into the end of the service pipe.

The operation of a siphon of the simplest type is as follows: In Fig. 76*a* the siphon is shown just ready to discharge. There are two volumes of water, separated by the air in the long leg, V, of the trap. As the pressure on every part of this confined mass of air must be equal to the hydrostatic pressure, and as there are

but two surfaces where the water is in contact with the confined air, it follows that the depth of water, C, in the tank must be the same as the depth, H, in the trap. When the depth, C, is increased, the water flows over the raised lip of the trap at D,

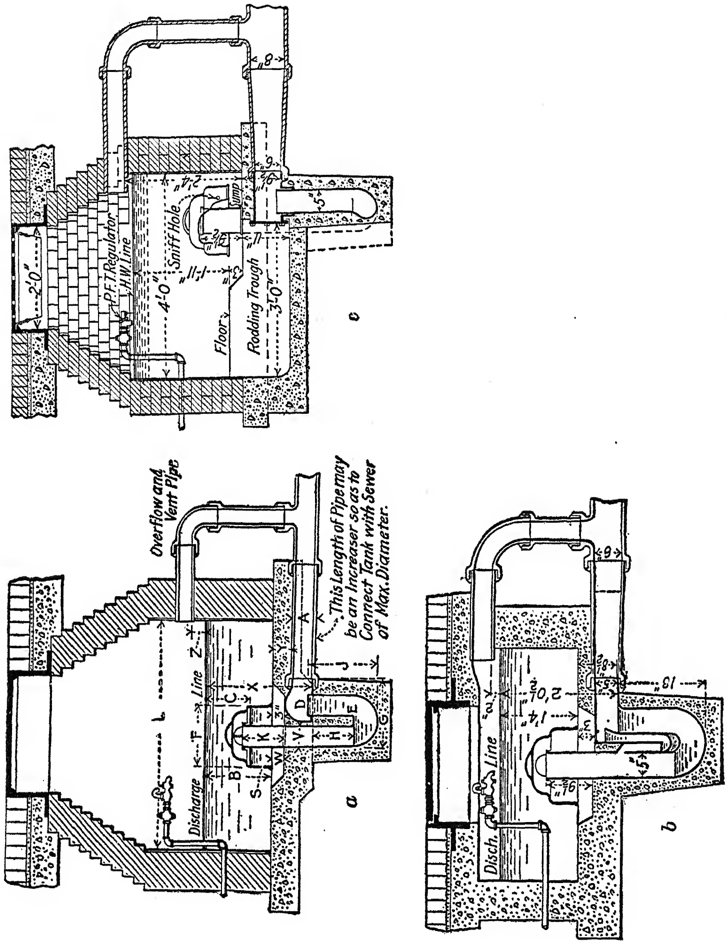


FIG. 76.—Types of flush-tanks. (*Miller.*)

this discharge allowing a little air to escape below the bend at E. The air pressure being released in this way, water passes up with a rush within the bell and into the long leg of the trap. When the water has been drawn down in the tank until its surface is below the sniff hole S, air rushes into the bell and slows up the siphonic action, but the discharge continues until the water

TABLE 45.—DIMENSIONS AND CAPACITIES OF MILLER SIPHONS, STANDARD SETTING (FIG. 76*a*)

Sewer diameter, inches	Average discharge rate, cubic feet per second	Bell diameter, inches	Trap width, inches	Trap depth, inches	Trap rise, inches	Minimum tank diameter, feet	Depth, inches, from discharge line to		Floor depth, inches	Rise to overflow, inches	Trap diameter, inches
A		*F*	*G*	*J*	*K*	*L*	Floor *B*	Invert *X*	*Y*	*Z*	*V*
4 to 6	0.35	13¼	14	14¼	8¾	3	14	22¼	4¼	3	4
6 to 8	0.73	16⅜	18¼	22¾	9½	3	23	34	5	2	5
8 to 10	1.06	20½	20⅜	29⅝	11	4	30	44	6	2	6
12 to 15	2.12	25½	27½	36½	13	4	35	51¼	6¼	2	8

TABLE 46.—DIMENSIONS AND CAPACITIES OF MILLER SIPHONS, SHALLOW SETTING (FIG. 76*b*)

A, inches	Average discharge, cubic feet per second	*F*, inches	*G*, inches	*J*, inches	*K*, inches	*L*, feet	*B*, inches	*X*, inches	*Y*, inches	*Z*, inches	*V*, inches
6 to 8	0.55	16⅜	21	18	9½	3	16	24½	3½	2	5
8 to 10	0.90	20½	20	19¼	11	4	19	39	6	2	6

TABLE 47.—DIMENSIONS AND CAPACITIES OF MILLER-POTTER SIPHONS (FIG. 76*c*)

A, inches	Discharge, cubic feet per second	*F*, inches	*G*, inches	*J*, inches	*K*, inches	Minimum *L*, feet	*B*, inches	*X*, inches	*Y*, inches	*Z*, inches	*V*, inches
6 to 8	0.73	16⅜	18¼	22¾	9½	3	23	34	5	2	5
8 to 10	1.06	22½	20⅜	29¾	11	4	30	44	6	2	6

reaches the bottom of the bell, where the water in the two legs of the trap at once forms a seal and the tank begins to fill again.

The elevation of the lip of the short leg at *D* above the bottom of the outlet is an important detail, as upon it the first sudden discharge of the trap seems to depend. In the older types of flushing apparatus this first strong flush was accomplished by using an auxiliary siphon at the bottom of the trap casting—a detail retained in the Rhoads-Miller siphon, Fig. 76*b*, for use where shallow construction is imperative.

The dimensions of the Miller apparatus, required by designers, are given in Tables 45, 46 and 47. The diameter of the tank is the minimum which is generally considered desirable for siphons of the size listed. The discharge is the average given by the makers for that size and setting of siphon.

The setting shown in Fig. 76*a* does not afford access to the sewer, but the Miller-Potter design shown in Fig. 76*c* affords this advantage. The manhole at the dead end of the sewer is provided with a flush tank and siphon, and while this is more expensive than the standard type, it affords an opportunity to insert a cleaning rod into the end of the sewer.

REGULATING DEVICES

One function of a sewage-flow regulator is to prevent the surcharge of an intercepting sewer by closing an automatic gate upon the branch sewer connection, thus cutting off the sewage and forcing it to flow to another outlet. Another function is to regulate the flow in time of storm so that discharge from one sewer carrying a heavily polluted storm flow may be admitted to the intercepter in greater proportion than that from another sewer carrying a more dilute storm flow.

A storm overflow is designed to allow the excess sewage above a definite quantity to escape from the sewer in which it is flowing, through an opening into a relief sewer.

Whenever an outlet is in a body of water which at times may rise above it, and it is necessary to prevent this water from entering the sewer, a backwater or tide gate is employed.

The purposes of the regulator and the storm overflow are supplementary, namely to cause the ordinary flow of sewage to be delivered to a distant point of discharge, and in time of storm to allow the excess flow to pass into a nearer watercourse.

145. Regulators.—A simple type of regulator, Fig. 77, has a cast-iron body which is bolted to the end of the main sewer and

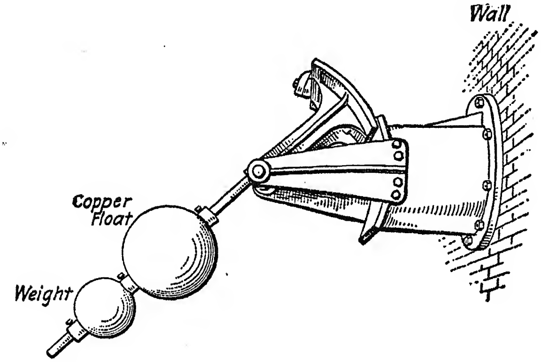


FIG. 77.—Coffin regulator.

projects into a chamber in which the sewage will rise to the same height as in the intercepter with which it is connected. The

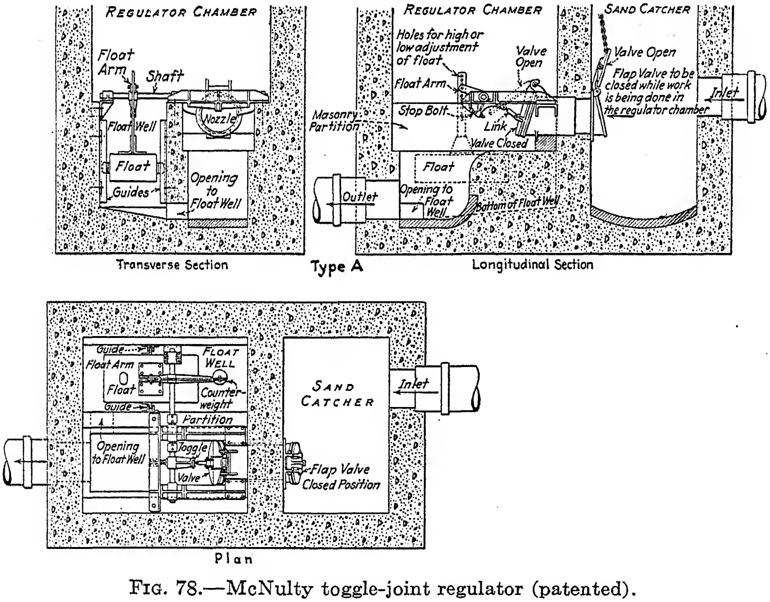


FIG. 78.—McNulty toggle-joint regulator (patented).

steel shaft carries an adjustable copper float and a weight by which the action of the device can be varied somewhat. Practically, it is difficult to keep this apparatus in operation, and

more complicated types of apparatus have been found to be more certain in action.

One of the best-known types of regulator is that manufactured by the McNulty Engineering Company of Boston, Fig. 78. This was developed for the connections between the Boston combined sewers and the intercepting sewers. The orifice in the main sewer is designed of sufficient capacity to allow the desired quantity of sewage to pass through it. In some cases it is necessary to provide a low dam in the main sewer immediately below the orifice to assist in diverting the sewage. The regulating gate seats against the orifice or nozzle. This gate is hinged and connected by a toggle joint with a lever which has a large float on its other arm. This float is in a well having a connection with the intercepting sewer, so that the rise and fall of the sewage in the intercepter raises and lowers the float. The apparatus can be adjusted so that the orifice will be closed when the float rises to any predetermined elevation.

OVERFLOWS

Storm overflows may be of the following types; overfall or side weirs, siphons, baffled weirs and leaping weirs.

146. Side Weirs.—An overfall weir is usually constructed in the side of a sewer, and the excess flow escapes over the crest when the elevation of the sewage is above that of the weir. One method of design of such a structure is described by W. C. Parmley[1] in a paper on the Walworth Run sewer in Cleveland. His study was primarily a mathematical treatment of the problem as there were no experimental data upon which it could be based, and later experiments have not borne out his assumptions.

The earliest experiments to determine the discharging capacity of side weirs that have come to the authors' attention were those of Hubert Engels of Dresden. From his tests he derived the formula

$$Q = \tfrac{2}{3}\mu\sqrt{2g}\sqrt[3]{l^{2.5}h^{5.0}}$$

Where Q = discharge over the side weir in cubic feet per second
l = length of weir crest in feet
h = head on lower end of weir in feet

[1] *Trans.*, Am. Soc. C. E., 1905; **55,** 341.

Substituting 0.414 for $\frac{2}{3}\mu$ as determined by his tests, this expression becomes

$$Q = 3.32\, l^{0.83} h^{1.67}$$

The discharge over the weir may be increased by contracting the channel as shown in *b* and *c*, Fig. 79. For the case of contracted channels, the formula becomes

$$Q = 3.32\, l^{0.9} h^{1.6}$$

This does not include a term for the ratio $b_o b_v$, and probably cannot apply to conditions where the contraction differs materially from that of the experimental channels.

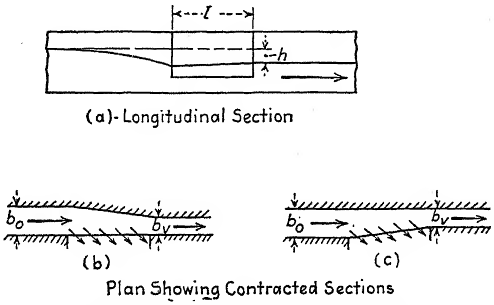


FIG. 79.—Discharge over side weirs.

The depth at the lower end of weir crest in *a*, Fig. 79, is seen to be greater than at the upper end, the explanation given being that the increasing depths on the weir crest result from the conversion of kinetic into potential energy along its length. This phenomenon is not reported by Coleman and Smith,[1] nor by Prof. Harold E. Babbitt[2] of the University of Illinois in their experiments on similar weirs, though the former state that the water in the flume rises downstream from the weir due to a decrease in velocity with depth. It did occur, however, in the large spillway structure in Tasmania, described by W. H. R. Nimmo,[3] and was observed by Tyler, Carollo and Steyskal[4] in experiments on side weirs at the Massachusetts Institute of Technology. In the latter experiments, the low point was about

[1] COLEMAN, GEORGE STEPHEN, and DEMPSTER SMITH: The Discharging Capacities of Side Weirs, London, 1923.

[2] BABBITT, "Sewerage and Sewage Treatment," 2nd Edition; John Wiley and Sons, Inc.

[3] Side Spillways for Regulating Diversion Canals. *Trans.*, Am. Soc. C. E., 1928; **92,** 1561.

[4] Discharge over Side Weirs with and without Baffle. *Jour.*, Boston Soc. C. E., 1929; **16,** 118.

one-third the weir length from the upper end of the weir, its position depending to some extent upon velocity in the main channel.

Babbitt experimented with weirs made by cutting out the sidewalls of 18- and 24-in. vitrified pipe sewers, with crests from 16 to 42 in. long. His results are expressed in the formula

$$l = 2.3vD \log \frac{h_1}{h_2}$$

Where

l = length of weir in feet.
v = velocity of approach in feet per second
D = diameter of the sewer (circular) in feet
h_1, h_2 = head on upper and lower ends of the weir.

In Engels' experiments, which he described in "Mittelungen aus dem Dresdener Flussbau-Laboratorium,"[1] discharges of from 0.52 to 6.35 cu. ft. per second in the main channel with 0.12 to 2.95 cu. ft. per second over the side weir were employed, the width of main channel varying from 0.67 to 6.56 ft., slope from 0.000091 to 0.001, and length of weir crest from 1.64 to 32.8 ft.

147. Transverse Weirs.—One method of avoiding the uncertainty involved in the use of side-weirs, employed at Louisville, Ky., is to place the weir directly across the line of flow and deflect the sewer to one side, thus reversing the arrangement used with side-weirs. It should be possible to determine the discharge of a weir in this position by the usual weir formulas with greater certainty than when the weir is in the side wall of the conduit, as the variation in head along the crest will presumably be smaller.

148. Baffled Side-weirs.—A structure planned to increase the discharge of a side weir is the baffled side-weir, illustrated in Fig. 80, which shows the Crouse Ave. connection to the intercepting sewer at Syracuse. This structure was built on the existing egg-shaped Crouse Ave. sewer by Glenn D. Holmes, Chief Engineer of the Syracuse Intercepting Sewer Board. A horizontal 4-in. concrete slab is placed across the sewer at an elevation which permits the desired maximum flow in the intercepter to pass underneath. A diversion wall set at an angle of about 40 deg. with the axis of the sewer diverts the flow above

[1] "Forschungsarbeiten auf dem Begiete des Ingenieurwesens," p. 200, 201; "Mitteilungen aus dem Dresdener Flussbau-Laboratorium, II," Ver. deut. Ing., 1920; **101**, 106.

this slab into a relief sewer depressed to allow for entry and velocity head losses.

In a later modification of the baffled side-weir Holmes omitted the horizontal cut-water slab and relied entirely on stop planks set in grooves in the sewer walls, to deflect the desired quantity of storm water. He used two 18-in. deflecting baffles set at an angle of 45 deg. with the direction of flow. The upstream baffle is 9 in. higher than the downstream baffle and comes into service at periods of maximum flow. The downstream baffle deflects ordinary excess flows.

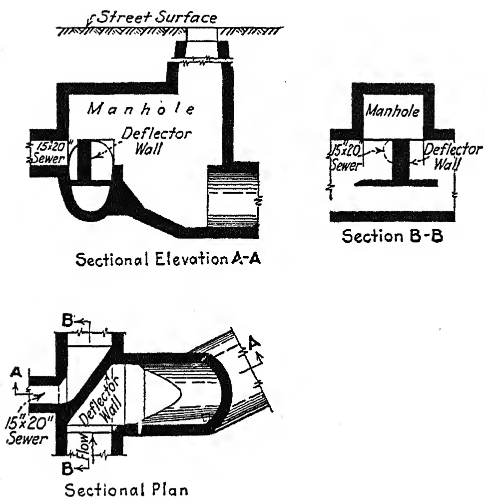


FIG. 80.—Baffled side-weir, Syracuse, N. Y.

In considering the hydraulic problems involved, the study must be largely theoretical, as there are few experimental data available upon which to base the design. If the area below the deflecting baffle be considered as an orifice, contraction will occur on the side in contact with the baffle. That this contraction may appreciably depress the surface of the sewage below and thus determine the elevation at which the baffle or skimmer should be set, is evident from experiments reported by Nimmo.[1] The increase in velocity due to pressure head should also be taken into account, together with the rise in level further downstream where the conditions of gravity flow in the sewer will have

[1] NIMMO, W. H. R.: Side Spillways for Regulating Diversion Canals. *Trans.* Am. Soc. C. E., 1928; **92**, 1561.

been reestablished. The elevation of the overflow weir and of the lower edge of the baffle may then be determined.

Experiments at the Massachusetts Institute of Technology by Tyler, Carollo and Steyskal in May, 1928, indicated that the horizontal skimming plate had no advantage, and also that a deflecting bulkhead placed squarely across the conduit, with its bottom edge at the elevation of the weir-crest, was more efficient in deflecting excess flow over a side-weir than one set at an angle. They suggested as a result of their experiments, that by making the elevation of the lower edge of this bulkhead adjustable, it could be set to pass the desired amount of sewage down the main conduit and would force much larger discharges over a side-weir of given length of crest, than could be discharged by a similar weir without the bulkhead. This increase in discharge is due to an increase in head on the weir caused by the change of velocity head to static head above the bulkhead. A condition of non-uniform flow exists at and below the bulkhead and a jump may be produced. A bend or a flat gradient downstream may force the jump back against the bulkhead, which increases the discharge over the side-weir. While the range of the experiments was not great enough to prove that these results would be obtained under all conditions, the indications are significant.

149. Siphon Spillways (Patented).—The siphon affords a means of regulating the maximum water-surface elevation in a sewer with smaller variations in high-water level than can be secured with other devices. It works automatically and without mechanism and, as it utilizes all of the available head, discharges at higher velocities than do overflow weirs. While this device has obvious advantages, its infrequent use is due, doubtless, to inadequate information concerning certain matters pertaining both to its design and operation, such as, for example, the minimum head required for priming, the possibility of odors where vents are required to remove air from the outlet chamber, or possible noise and vibration from sudden starting and stopping of the siphon, especially with high heads. Fig. 81 shows a siphon relief structure which was installed in 1926 at the end of the intercepting sewer at Akron, Ohio. The purpose of this structure is to prevent the water level below the coarse racks from rising above the elevation at which the flow to the treatment plant is just equal to 93.6 million gallons daily, the design capacity of the

plant, or, in case it becomes necessary to shut down the treatment plant, to remove the entire flow of the intercepting sewer. The sewer system of Akron is on the combined plan and the capacity of the intercepter is 252 million gallons daily. The siphon reliefs, four in number, are each designed for about 65 million gallons daily capacity.

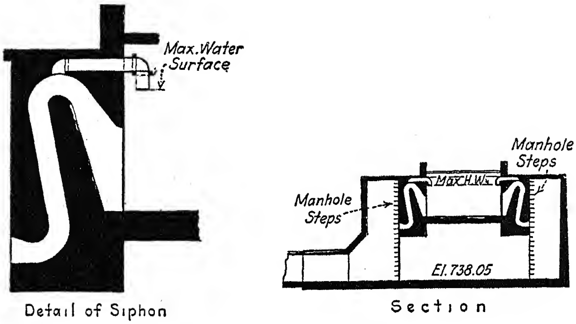


FIG. 81.—Siphon relief structure on intercepting sewer, Akron, Ohio (patented).

The approximate cross-section for the siphon throat can be determined by the formula

$$Q = ca\sqrt{2gh}$$

Where Q = discharge in cubic feet per second
c = coefficient of discharge (0.6 to 0.8)
a = area of cross-section of throat in square feet
h = head in feet = difference in water level above and below siphon.

A trial section may then be drawn, the losses determined for this section, and the corrected values of Q or a computed. The design then involves working out such details as the method of venting, shape of section to meet the particular requirements and the fixing of elevations of inlet, spillway, outlet, and air vent. An air vent with area equal to $a/24$ has been found to be ample. The siphon inlet should be large so that the velocity at entrance will be small, thus preventing large losses of head. The section tapers gradually to the throat, and the lower leg, which may be either vertical or inclined, is of uniform section or slightly flared. The outlet is so constructed that air bubbles formed during priming will be carried out of the siphon.

When the water rises to the elevation of the overflow in the siphon, it will flow over the crest in a thin sheet, falling against the opposite wall and carrying with it some air. This condition will continue until the water has risen sufficiently to seal the air vent, when the air remaining in the siphon will be exhausted quickly and siphonic action will begin. If the capacity of the siphon is greater than the amount of excess flow in the sewer, the water will be lowered until the air vent is uncovered and siphonic action will cease. Generally it is desirable to have the elevation of the air vent nearly the same as that of the overflow, so that there will be but little overflow before the siphonic action begins or after it ceases. In some cases it may be desirable to have the air vent lower than the overflow, so that there will be no overflow after the siphon breaks. In this case, siphonic action will begin almost immediately after water begins to overflow the crest. The maximum operating head, theoretically, for a siphon is about 33.9 ft. at sea level, decreasing at the rate of about 1 ft. for 850 ft. increase in elevation above sea level. This is more than ample for such uses as may be found for siphon reliefs on sewer construction. In a paper on "Siphon Spillways" by G. F. Stickney[1] it is stated that the efficiency of siphons which have been constructed for reservoir spillways is from 60 to 70 per cent. Tests on a model at the hydraulic laboratory of the Massachusetts Institute of Technology gave a coefficient of discharge of about 0.54, while Weirich (France, 1917), also working with a model, obtained a coefficient of 0.97. The reason for the unusually high efficiency obtained by Weirich is not given in the published report, and similar efficiencies have not been attained elsewhere.

150. Leaping weirs may be provided above openings in the inverts of sewers, so that the ordinary flow of sewage falls through the openings and passes to the intercepters. At times of storm, the increased velocity of flow causes most of the sewage to leap the openings and pass on down the sewers to the storm outlets.

In one branch of the intercepting sewer system of Syracuse, N. Y., leaping weirs were used at the connections of existing sewers with the intercepters, and float regulators were also

[1] *Trans.*, Am. Soc. C. E., 1922; **85**, 1088.

employed to safeguard the intercepters against surcharge. The type of weir employed at Syracuse is shown in Fig. 82. It is

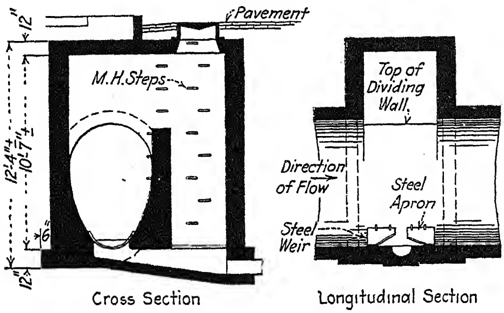


FIG. 82.—Leaping weir at Syracuse, N. Y.

formed with a steel weir plate inclined upward so as to give a spouting effect, and a movable steel apron to catch the discharge at times of high flow. During dry weather when the quantity of sewage is small and the velocity slight, the sewage drops over the weir into the channel leading to the intercepter. At times of storm flow, the increased velocity causes the sewage to leap the opening. Several of these weirs have been in use for over 15 years, and have given good satisfaction.

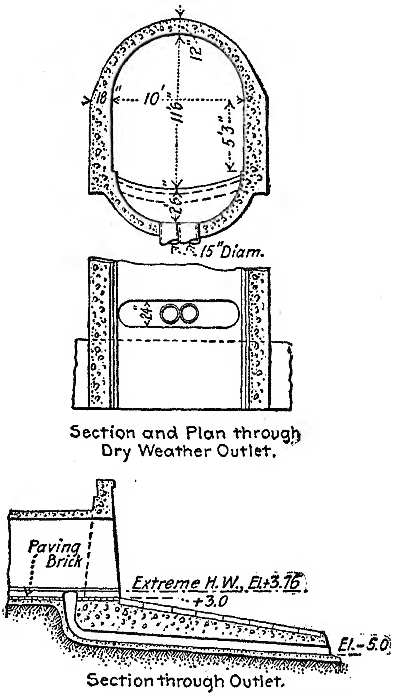


FIG. 83.—Duplex outlet, Minneapolis.

151. Outlets.—Where sewage is discharged into tide water, a lake or a deep river, the outlet is usually submerged to a considerable depth in order to disperse the sewage thoroughly through a large volume of water before it rises to the surface. For example, the 66-in. steel outfall sewer of Rochester, N. Y., laid in 1913 in Lake Ontario, terminates in

a timber crib 7,000 ft. from the shore. The outlet is 10 ft. above the bottom of the lake, which is about 50 ft. deep at this place. Sometimes it is advantageous to allow the storm flow in a combined sewer to discharge freely, above high water elevation, although the dry-weather flow should be discharged below the water surface when the stream receiving the discharge is at its lowest elevation. Fig. 83 illustrates a duplex outlet built at Minneapolis to meet these conditions. The outlet is on a bank of the Mississippi River, where the difference between the elevations of flood and dry-weather stages is great. Two 15-in. cast-iron pipes run out below the paved apron of the storm-water outlet and discharge the dry-weather sewage 5 ft. below the low-water level of the river. The invert of the storm-water outlet is 9 in. below the high-water level in the river, so that the sewer will have a free discharge at all times.

152. Tide Gates.—A tide gate or backwater gate is now generally made of a leaf or shutter of planks, hinged at the top and resting against an inclined seat when closed.

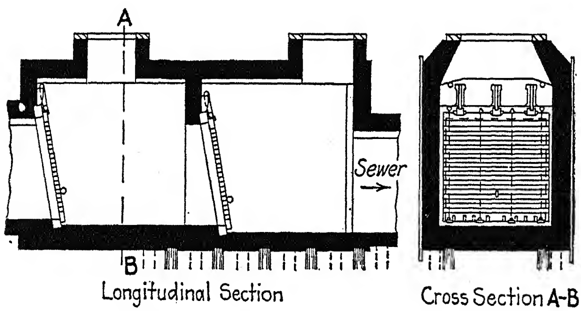


FIG. 84.—Tide-gate chamber, Boston, with McNulty gates (patented).

The type of tide gate and chamber now used in Boston is shown in Fig. 84. The seat is a heavy wooden frame with which the leaf makes a tight joint, aided by a rubber gasket slightly recessed along each edge, so that the nails holding it to the wood do not project and prevent the proper compression of the rubber when the gate is subjected to back pressure. In the case of smaller sewers a cast-iron frame is inserted in the masonry to form a seat for the gate. Small sewers are sometimes provided with flap-valves of cast iron for tide gates.

PUMPS AND PUMPING STATIONS

In the design of sewage works, it may be necessary to provide for pumping where the sewage or storm water is collected at so low an elevation that discharge by gravity is impossible, as at Washington; to reach a desirable treatment site, as at Baltimore; to lift the sewage from areas too low to drain into the main system by gravity; or to force water into streams or tidal inlets receiving sewage, which would become offensive unless flushed in this way.

Whether the sewage shall be lifted at one or more points is usually a matter to be settled by comparing the fixed and operating expenses corresponding to different plans. The operating expense of raising all of the sewage at one point is less than that of pumping at two or more points. On the other hand, if all of the sewers are made to drain by gravity to one place, their cost may be greatly increased on account of the deep cuts and large cross-sections necessary in order to obtain satisfactory velocities of flow. Various projects must often be considered, both with and without pumping, and the extra cost necessary to drain to one point, together with the cost and the capitalized annual charge for operation and depreciation of the pumps, must be compared with similar charges for a project with two or more stations. (See Chapter XXI.)

The requirements for pumps to be used in elevating sewage or drainage water differ considerably from those for pumping clean water. The lift is comparatively small; simplicity and reliability are of more importance than high efficiency; valves (if required) and passages must be able to pass solid matter with a minimum of clogging or repairs; and the possibility of automatically starting and stopping pumps to take care of variations in flow is often a controlling factor.

153. Types of Pumps.—There are four types of pumps which may be used for sewerage and drainage work, namely, reciprocating pumps, centrifugal pumps, screw pumps, and pneumatic ejectors.

154 Reciprocating Pumps.—As the name indicates, a reciprocating pump is one in which water is displaced by the reciprocating motion of a piston or plunger within a cylinder. It is a *positive-displacement* pump, and each stroke of the plunger or piston necessarily displaces a definite volume of water. The actual quantity pumped per stroke is less than the displacement

by the amount of the *slip*, or quantity leaking through the valves and past the piston.

The use of reciprocating pumps for sewage or drainage water necessitates:

(*a*) Screening of the sewage for the removal of solids large enough to clog the pumps.

(*b*) Settling the sewage to remove grit which would cause rapid wear.

(*c*) A design of pump with large passages and with valves capable of passing the smaller solids without clogging; and providing for repacking and for taking up wear without too great difficulty.

A considerable variation in the speed of a steam pump is possible, with a corresponding variation of capacity, and changes in the sewage flow can be cared for in a measure by regulating the speed; but a pump driven by electric motor or internal combustion engine generally must be operated at a constant speed, and changes in the flow must be met by changing the number of pumps in operation, unless it is possible to equalize the rate of flow by interposing a large suction well or reservoir.

Pump valves for sewage are either of the ball or of the flap type. Ball valves are rarely used except for sludge. Flap valves, which are generally used in sewage pumps, are either actually hinged, or attached to the valve deck so as to move as if they were hinged. Flap valves frequently cause much trouble because sticks and rags are caught on their seats and hold them open. Hinged flaps are more often used than the simple flap pattern and have leather or rubber disks held between metal plates, the top plate having an arm running sideways to a hinge connection with the valve seat.

Reciprocating pumps for sewage are now rarely installed. In earlier years they were employed in a number of large plants because of the high duty which could be obtained in conjunction with triple-expansion or compound steam engines; but recent improvements in centrifugal pumping machinery have made it very nearly as economical as the best steam pumping engines. The latter are much more costly, occupy far greater space, and require more attendance for their operation, than do centrifugal pumps.

There are several large sewage pumping stations, as at Boston and Baltimore, for example, in which high-duty reciprocating

pumps were installed years ago, which are still operating satisfactorily and economically.

155. Centrifugal Pumps.—A centrifugal pump consists of two principal parts, a fixed *casing*, and a rotating *impeller* or

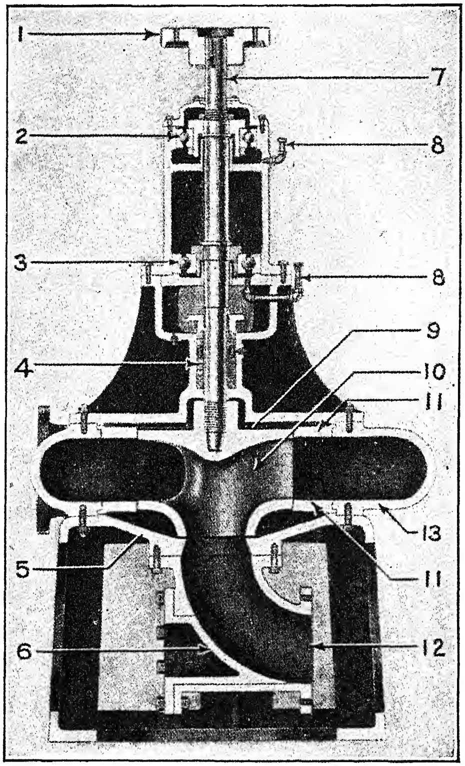


FIG. 85.—Section of vertical centrifugal pump for sewage. (*Fairbanks-Morse.*)

This section of a sewage pump shows how the impeller (10) and the flexible coupling (1) are mounted on the shaft (7) which is carried by a combined radial and thrust bearing (3) and guided by a radial bearing (2). Where it passes through the stuffing box, the shaft is protected by a bronze sleeve (4). The bearings run in oil the level of which is indicated by the gauges (8). The suction elbow (12) with a clean-out hole (6) is attached to the lower or suction head (5) which is cast as part of the base. The volute casing (13) which may be equipped with adapter rings (11) for impellers of various diameters supports the upper head (9) and bearing housing.

runner provided with one or more *vanes*. Water is admitted to the impeller near the center or eye, either on one side (*single suction*), or on both sides (*double suction*). The vanes may be held between side plates which form part of the impeller (*closed impeller*), or the plates may be omitted or fastened rigidly to the shell of the pump, so that the impeller consists substantially of

the shaft and vanes only (*open impeller*). When the impeller is rotated the water is thrown off at the periphery with a considerable velocity; and as the water is confined within the casing (*volute*), the velocity is largely transformed into pressure.

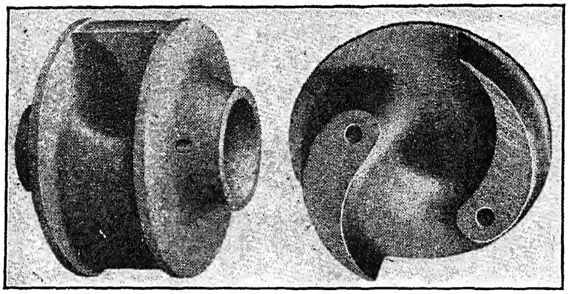


FIG. 86.—Impeller used in Fairbanks-Morse sewage and trash pumps.

It is impracticable to build up sufficiently high velocities with a single impeller to overcome very high heads; but any lift can be pumped against by arranging a number of pumps in series, each taking its suction from the discharge of the preceding

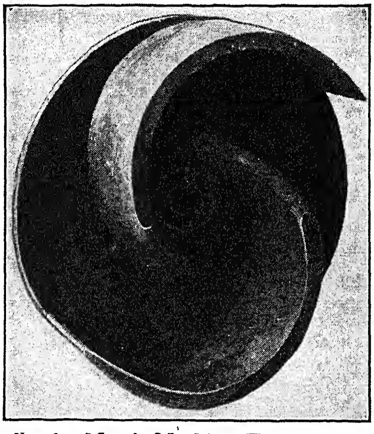


FIG. 87.—Impeller for Morris Machine Works non-clogging pump.

pump. When clean water is to be handled, two or more impellers can be mounted on a single shaft, and if the casing is properly shaped to guide the water from one impeller to the next, a *multistage* pump is obtained. Such a pump is not suited to handling sewage, but it is seldom that heads are high enough in

sewerage work to require more than a single stage. Where comparatively high heads are involved, two or more separate pumps can be arranged in series.

The early centrifugal pumps were crude and inefficient. Even after considerable efficiency had been obtained in pumps for handling water, which were furnished with closed impellers and guide vanes, it was still necessary to use open impellers for sewage in order to reduce the clogging to a minimum, which also

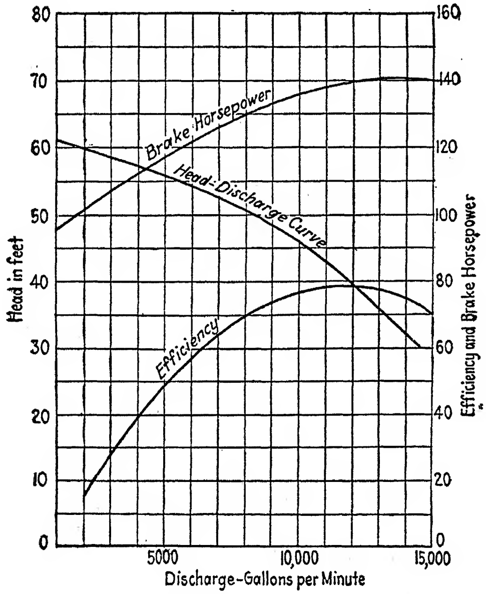


FIG. 88.—Characteristic curves of Worthington 20-inch vertical volute trash pump.

reduced the efficiency. In recent years, however, great improvements have been made, especially in the design of impellers of the "nonclogging" or "trash pump" type, and it is now possible to obtain very good efficiencies in pumps which are remarkably free from clogging, even when handling unscreened sewage.

The size of a centrifugal pump is defined as the diameter of the discharge pipe.

Centrifugal pumps are called vertical or horizontal, depending upon the position of the shaft. A section of a vertical cen-

trifugal pump is shown in Fig. 85. Figs. 86 and 87 illustrate forms of so-called "nonclogging" impellers especially designed for handling sewage and drainage water.

A very important feature of the centrifugal pump's characteristic (for constant speed) is that the brake horsepower required by the pump usually increases as the head decreases, at least for quite a range, as shown in Fig. 88. This is due to the greatly increased quantity handled by the pump at the lower head, sometimes at considerably less than maximum efficiency. For this reason it is important to carry out the horsepower curve beyond the point of maximum lift, in order to know what load might be put on the motor by the greatly increased discharge which would result from a broken discharge pipe or an unusually high suction level. Commonly used designs of sewage pumps will show a maximum horse-power not much greater than the horsepower required at the best operating point, and it is customary to put on a motor large enough to take care of this maximum demand. This is important in sewage pumps because the time of high suction level (resulting in low total head) is likely to be the time of storm or other emergency condition when interruption of operation would be most serious.

156. Screw Pumps.—The impeller of a screw pump is similar in form to the propeller of a ship. Its rotation causes the water to move axially or along the shaft. Such an impeller may be installed in a casing or shell which is practically a section of pipe.

The screw pump is most advantageous in handling large quantities of water when the lift is small, not over 15 or 18 ft. Its principal application has been for handling large volumes of storm water, as at New Orleans.

The screw pump usually has the desirable characteristic that with decreased head the capacity increases without correspondingly increasing the power required by the pump after passing the best operating point. Therefore, with high water on the suction side or low overall head, the pump is no less dependable than under normal conditions.

157. Pneumatic Ejectors.—A pneumatic sewage ejector consists of an air-tight iron receiver with check valves on the inlet and outlet pipes, provided with a float-controlled valve or switch by the action of which compressed air is admitted to the receiver when the latter is filled, thus expelling the sewage.

In municipal sewerage works, ejectors are sometimes used to lift the sewage from small low-lying districts. They have also been used extensively for pumping sewage from deep basements into city sewers.

One of the earliest types of pneumatic ejector is the Shone, the present form of which is illustrated in Fig. 89. This ejector is now made by Yeomans Bros. Co. Its operation is extremely simple. Sewage enters the receiver through the check valve

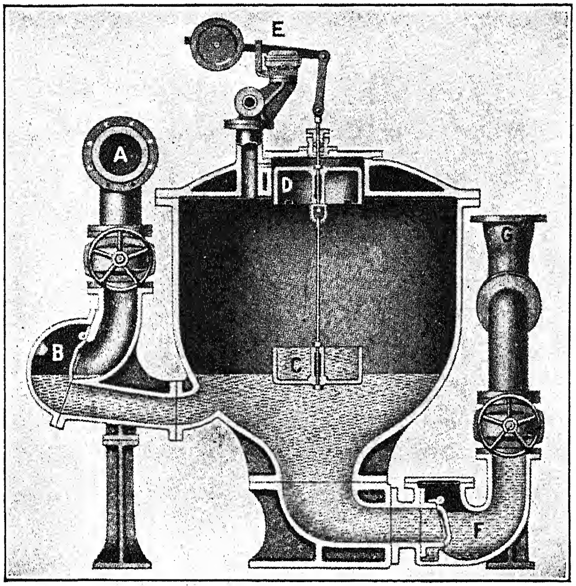


Fig. 89.—Shone sewage ejector.

B, and accumulates until it lifts the inverted cup *D*, with the aid of the flotation effect of the entrapped air; this closes the exhaust valves and opens the compressed air valve. The air then flows into the receiver and forces the sewage through the check valve *F* and into the discharge pipe, until the level of the liquid falls to such a point that the weight of cup *C* and the water it contains causes the cup to drop and reverses the air valves. While the ejector is discharging, the incoming sewage is retained in the inlet pipe, above check valve *B*, or is being discharged into another receiver. Fig. 89 shows the ejector discharging but nearly empty. The operation will be repeated as long as sewage flows

to the receiver and the supply of compressed air is maintained, if the seating of the check valves is not prevented by solid matter.

These units discharge intermittently and consequently are usually installed in pairs and interlocked so that one discharges while the other is filling. This gives a more nearly uniform rate of flow in the discharge pipe and thus permits the use of a smaller pipe.

The term *sewage ejector* is not limited to pneumatic ejectors, but is used also to include any type of apparatus suitable for the same service. *Electric centrifugal ejectors* are in common use. They are, in effect, small automatic pumping plants, including receiving tank, vertical centrifugal pumps, electric motors, and float-controlled switches.

158. Pumping stations have been classified in a variety of ways, such as according to capacity or nature of prime movers. For a detailed discussion or description of pumping stations, the student is referred to Vol. I, "American Sewerage Practice," or to books dealing more particularly with this subject. Examples of pumping stations are here given to indicate the methods of pumping sewage now in use.

Fig. 90 illustrates a type of pumping plant frequently used for small installations. The equipment is automatic in operation and the ejector may be either of the centrifugal or pneumatic type. The building in which the equipment is housed consists of a dry well for receivers and pumps, while the motors or compressors may be placed on an upper floor in a superstructure of suitable size and appearance, or the entire equipment may be assembled below the street level, without superstructure of any kind and with admission to the dry well through manholes.

Ward Street Pumping Station, Boston (Metropolitan Sewerage Works).—This station, as originally constructed, is typical of the best practice of its date, when large high-duty pumping engines were employed. It was built in 1902–1904. The buildings are of brick with granite trimmings, and include an engine room 65 by 120 ft. for three 50,000,000-gal. pumping engines, of which two were provided; a boiler room 38 by 105 ft., for four vertical boilers; a coal pocket 39 by 52 ft., a screen chamber[1] 42 by 47 ft., and a chimney 150 ft. high with a 6-ft. flue. Fig. 91 shows plan and section of the station. The pumping engines were built by

[1] The screens at this station are described in "American Sewerage Practice," Vol. III, 1st Ed., 321 *et seq.*

the Allis-Chalmers Company, and included vertical triple-expansion steam engines with cylinders 21, 38 and 58 in. in

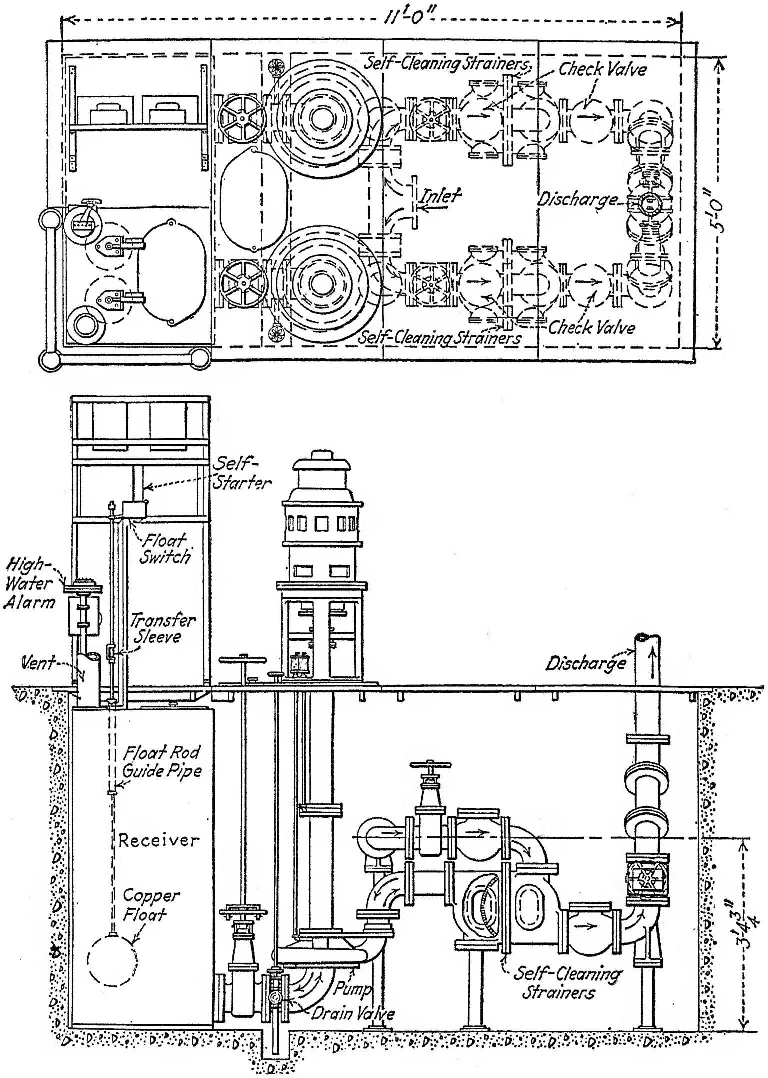


FIG. 90.—"Flush-kleen" dry basin sewage ejector. (*Chicago Pump Co.*)

diameter, with stroke of 60 in.; and there are three outside packed single acting plungers 48¼ in. diameter and 60 in. stroke. The fly wheels, two for each engine, are 18 ft. in diameter, and

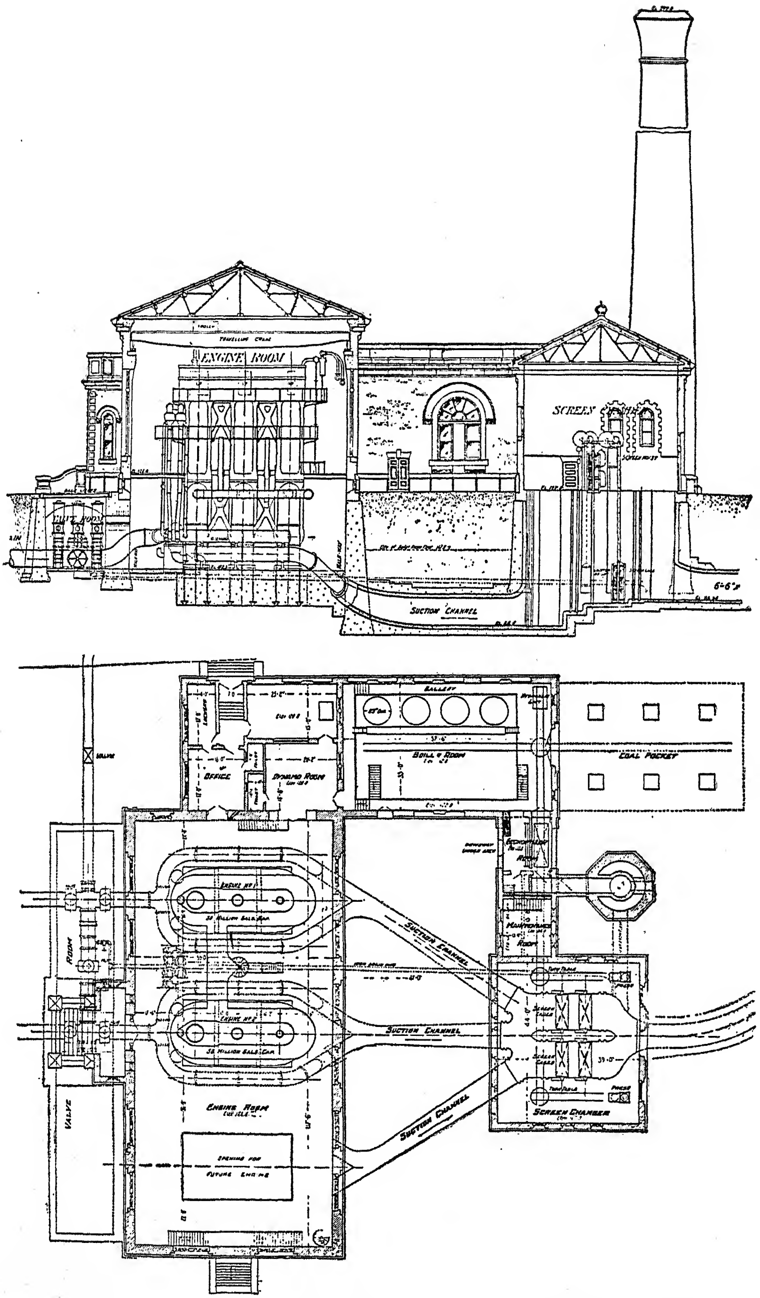


FIG. 91.—Ward Street pumping station as originally built (Boston. Mass).

the speed is 25 r.p.m. for a capacity of 50,000,000 gal. per day. The lift is 45 ft. Each suction and delivery chamber contains 36 valves, making a total of 432 valves for each engine. The nominal area of the waterway through the suction and discharge valves is about 200 per cent of the area of the pump plungers. Upon duty trial in 1906 these engines developed a duty of 152,-700,000 ft.-lb. per 1,000 lb. of dry steam. Steam pressure was 150 lb. per square inch and slip was 3 to 3.6 per cent.

Although these pumping engines have proved satisfactory and are still in use, nevertheless, when a third pump was required, instead of providing another of the same type as contemplated in the original plans, a centrifugal pump was selected.

> The station was originally put in service in 1904. Since that time the design of centrifugal pumps has been so improved that it has been considered best to substitute one of this type for the plunger pump originally intended. The cost of the centrifugal unit as compared with the plunger type unit is at present probably in the ratio of about 1 to 6.[1]

The unit selected consists of a 36-in. Morris Machine Works centrifugal pump with a capacity of 50,000,000 gal. per day against a total lift of 45 ft. (of which 14 ft. are suction lift), directly connected to a Nordberg uniflow steam engine of 540 hp., and running at 150 r.p.m., with a guaranteed duty of 103,-000,000 ft.-lb. per 1,000 lb. of dry steam. This is used as a standby unit as its lower duty makes its operation less economical.

North Side Pumping Station, Sanitary District of Chicago. The sewage pumping station at the North Side Treatment Plant of the Sanitary District of Chicago is shown in Fig. 92. Five pumps for pumping sewage from the intercepting sewers to the treatment plant are provided, two having a rated capacity of 150 cu. ft. per second each and three of 100 cu. ft. per second each against a total dynamic head of 44 ft. They are of the horizontal centrifugal type, driven by direct-connected synchronous motors of 1,000 and 700 horse-power respectively, and are set in a pit directly over the suction channel. Space is provided for a sixth unit. The pumps are preceded by coarse screens of the vertical lift type with 4-in. clear openings, and discharge into individual conduits merging into a 12-ft. influent conduit leading to the grit chamber. For protection against flooding, duplicate overflow siphon spillways are provided in the influent

[1] *Report*, Metropolitan District Commission, 1924.

conduit which discharge back into the suction channel. Keeping in mind that this station has about twice the capacity of the Ward St. station, it is readily apparent how much more compact is the centrifugal pumping layout.

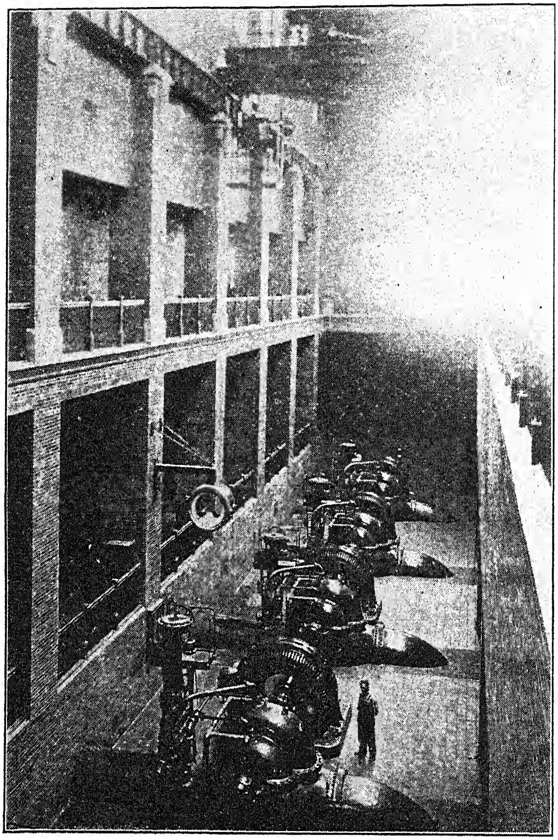


Fig. 92.—Sewage pumps at North Side Treatment Plant of the Sanitary District of Chicago.

Problems

1. Determine the elevations of the pipe inverts and weir crests of the siphon inlet chamber, Fig. 73, using the data given in the text, assuming the elevation of the 30-inch sewer invert as 100.00 ft.

2. Determine the conditions under which each pipe in the inverted siphon in Fig. 74 comes into service and compute all head losses for the following assumptions. Total length of siphon between manholes = 75.0 ft. Maxi-

mum discharge of 30-inch sewer ($S = 0.001$) = 13.2 c.f.s. Total fall available = 0.70 ft.

3. A 5.0 ft. circular combined sewer of concrete ($n = .013$) on a slope of 0.0005 will be full for the maximum discharge expected. It is desired to provide capacity for the maximum dry-weather flow of 10.0 c.f.s. plus an equal amount of storm water, and to divert the remainder over a side-weir into a storm-water outlet. Determine the length and height of side-weir required.

4. Design a siphon spillway for the same conditions of flow as in Problem 3 with $C = 0.70$ and 5.0 ft. available head. Draw plan and elevation showing siphon details.

5. Determine sizes of duplicate pumps and receiver for a pumping plant similar to that shown in Fig. 90, to handle an average flow of 10,000 g.p.d. with hourly fluctuations as in Fig. 8 average curve.

CHAPTER VII

PRELIMINARY INVESTIGATIONS, SURVEYING, AND INSPECTION

INVESTIGATIONS PRELIMINARY TO DESIGN

It is desirable to make comprehensive preliminary investigations of the district to be sewered, not only to obtain the data needed by the designing engineer and contractor, but also to place on record authentic information as to the local conditions prior to the construction of the system, which may be affected by it. Such information may be helpful in meeting future claims for damages. Contractors are justified in making lower bids when they are supplied with complete information about the conditions they will encounter than when they must estimate many of them. All facts ascertained, whether advantageous or not, should be made public, because the judgment of the engineer must be entirely unbiased and the contractor should in all fairness have all the information available to assist in formulating his bid. Also if any facts are suppressed, contractor's claims for extras based upon such facts may be allowed by the courts.

At the outset of the engineering work an attempt should be made to obtain all *maps* which furnish information about the district. City engineers, town and county surveyors, assessment boards, land title and insurance companies and public utility officials often have such maps and will permit copies of them to be made. In large sewerage undertakings, the maps prepared by the U. S. Geological Survey and the Post Office Department may be useful and occasionally the Bureau of Soils of the U. S. Department of Agriculture is able to supply helpful maps. All errors discovered in any of the maps should be marked on them so that later there can be no question about the errors in case these maps and uncorrected copies of them are used as evidence.

159. Field Work.—Unless satisfactory maps are available, *surveys* must be made. The degree of precision required will depend on the conditions of the problem; in large, important work it is sometimes advisable to establish a preliminary triangu-

lation system. The surveys should furnish the precise location of street and property lines, street railways, public parks and buildings, ponds, streams, large ditches and other features and structures which may influence, or be influenced by, the sewerage system.

An accurate, permanent and complete system of *bench levels* should be established throughout the area to be covered by the proposed sewerage system. In heavy work a bench mark should be established on each block of every street in which a sewer is to be laid, and at frequent intervals where topographic details are to be obtained subsequently. *Profiles* should then be run on all existing streets and alleys, and if the existing and "established" grades are different, notes about the latter should be obtained. This work should be extended to cover the district within which sewers may be needed during the next 30 to 50 years. Topographic notes should be obtained for the plotting of a map with contours at intervals of 1, 2, 5, or 10 ft., according to the configuration of the ground. The elevations of the beds of streams, ditches, canals and culverts should be ascertained and the maximum and minimum flow in them determined as accurately as practicable.

While the surveys are in progress, full notes of all existing *structures* should be obtained. The elevations of the sills of buildings and the approximate depths of their cellars should be ascertained. The character, age and condition of the pavements of streets in which sewers will be laid should be recorded, and the areas of all roofs, sidewalks and paved private alleys and drives should be obtained. All outlets of public and private sewers and drains and the discharge from each should be noted. *Existing sewers* should be investigated sufficiently to check the accuracy of the records of them, and if accurate records are lacking a survey of the sewers should be made. This is done by finding the location and elevation of the invert at each manhole. The condition of the sewers can be ascertained by illuminating their interiors with the aid of flashlights, or of two small mirrors; one is held at the top of a manhole so as to reflect sunlight to a mirror held by an inspector at the bottom of the manhole, who directs the light into the sewer to be examined. All available information regarding the location of water and gas mains, electric conduits and other underground structures should be obtained, and where such information is lacking it may be

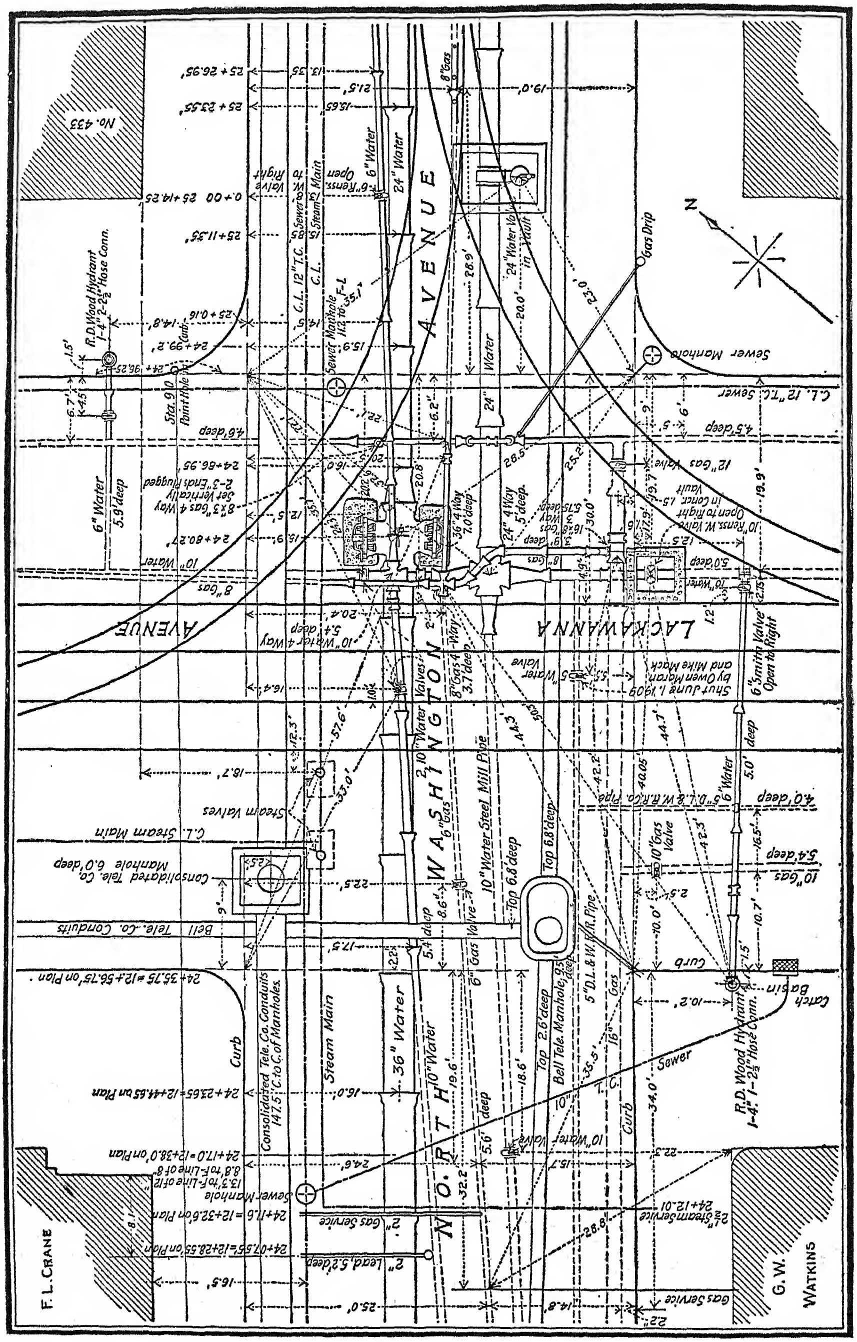


FIG. 93.—Part of street substructure map, Scranton.
(The original map is a little more than twice the size of this reproduction.)

advisable to dig pits in the streets to supply it. Fig. 93 indicates the numerous underground structures and pipes sometimes encountered in city streets. It is obvious that information as to the location of such pipes is needed before constructing additional sewers.

All local *rainfall* and *run-off* data should be collected, or where these are inadequate, measurements in the field should be undertaken if practicable. All information which builders and contractors can supply regarding *ground water* should be recorded, and in the case of low-lying land it may be desirable to sink pits to find out what the ground water conditions are.

The *character of the soil* in which the sewers must be constructed should be ascertained in order to enable the cost to be estimated with fair accuracy. For investigations of shallow depths, a sounding rod is often used. This is a steel rod or a pipe with a steel point and driving head. By placing the ear close to the rod as it is driven, it is possible, after a little practice, to estimate when the point passes into and through clay, sand, gravel and other earthy materials. When the rod strikes rock it rings. A post auger is often used to obtain samples of earth from shallow depths. For greater depths, well-sinking outfits are used, particularly in earth, and core drills are sometimes, although rarely, employed where it is desirable to obtain true samples of all underlying strata, or in other words, a vertical section of the material to be traversed.

Complete information concerning the local *wages* of unskilled and skilled labor and the cost of *construction materials* and supplies, and cost of construction of similar work previously done, should be ascertained. *Freight rates* should be looked up and also the rental charges for teams, trucks and equipment. This information is an aid in the preparation of reliable estimates.

160. Office Work.—Work on maps and profiles should be carried on as soon as practicable after the field work, so that studies preliminary to design may be started before the field work is finished. As a rule, maps on a scale of 200 ft. to 1 in. are large enough to permit all structures to be shown in adequate detail, but where there are many subsurface structures a scale of 100 ft. to 1 in. may be necessary for clearness. In Fig. 93 showing a street intersection in Scranton, Pa., the large amount of detail sometimes required on such maps is indicated. The

maps will usually require more than one sheet, and in such cases a key map should be drawn showing the way the detailed maps fit together. The maps should show the location of all street railways, railroads, buildings, pipes, conduits, manholes, gate boxes, catch basins, and the names of streets, parks, public buildings and water courses. The magnetic or true north, or both, should be indicated. It is very desirable to have the sewerage map on a single sheet, as the relations of the different parts of the system are thus shown most clearly.

A profile of each street in which a sewer is proposed should be plotted as in Fig. 53 p. 169. A vertical scale of 4 ft. to 1 in. and a horizontal scale of 40 ft. to 1 in. are commonly used. On these profiles should be plotted all information obtained from borings and soundings and the location of all pipes and conduits intersecting the proposed sewer trench. The names of all cross streets, the kinds of pavements, the elevations of pipes and conduits running along or crossing the street, and the depth of all deep adjacent cellars should be shown on these profiles.

When the construction is finished, *record maps and profiles* should be drawn to show clearly and locate accurately every detail of the system, so that there may be no uncertainty in the future concerning the works as actually built.

161. Contract Drawings and Specifications.—Detailed contract drawings should be completed before bids are requested, so that all of the data will be made available to prospective bidders. Such drawings should show, so far as practicable, all available information bearing upon the character of materials to be excavated, the location, size and character of structures likely to be encountered in the excavation, together with details of the works to be constructed.

One of the contract drawings for the Emerson Street sewer at Louisville is shown in Fig. 94. This sewer is in an unpaved street so the ground surface is very irregular and there is less detail on the plan than where there are curb lines or numerous underground structures to be shown. The data usually required for preparing contract drawings are illustrated by this figure and the method of showing the data, with scales, may be seen. By comparing with the profile in Fig. 53, the greater amount of detail and the difference in scale and arrangement which this necessitates, is apparent. Note that both a plan and profile are required for indicating the work to be done. The water shown

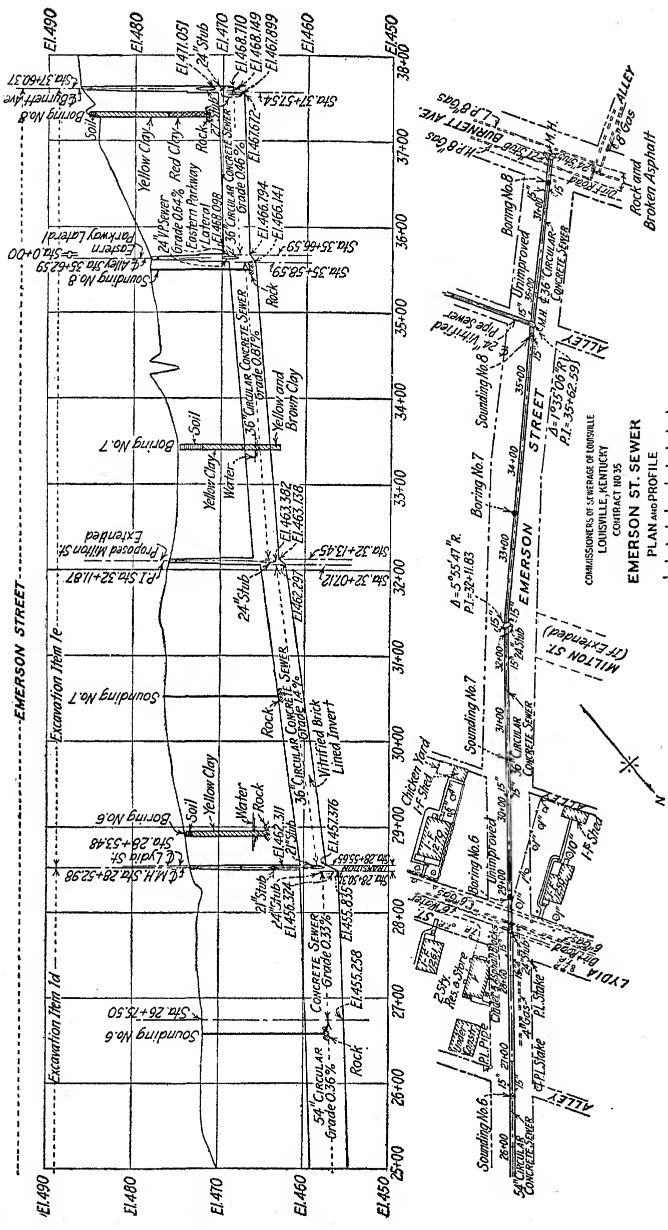


FIG. 94.—Contract drawing for sewer at Louisville.

in test borings gives the elevation of ground water as encountered at the time of making the boring.

The contract and specifications should be so drawn as to set forth clearly, and as completely as possible, all work, requirements and conditions included in or affecting the contract. While these details increase the engineering cost of the work, the net cost of the project will usually be less than where the drawings merely show in a general way what is to be done, or where the specifications are incomplete or obscure.

ENGINEERING WORK DURING CONSTRUCTION

162. Laying Out Trench Work.—There are a number of different methods in use for staking out trenches for sewer construction. In undeveloped country stakes may be driven on the center line of the sewer at 50-ft. intervals, from which the line of the excavation may be marked off on either side, after which the stakes are removed as the excavation proceeds. A similar method may be used in city streets, except that in place of the stakes large spikes are driven into the pavement. In order that these spikes may be readily found, a piece of heavy canvas about 4 in. square may be placed on the ground and the spike driven through it, thus pinning the canvas to the surface of the street.

Stakes or spikes may be driven on a line parallel to, and offset a few feet from the center line. In choosing the side on which these offset stakes should be driven, consideration must be given to the side of the trench on which the excavated earth will be thrown out, the stakes being placed upon the opposite side.

163. Reference Marks.—In staking out a sewer trench, the lines should be so referenced that they may be located readily and quickly at any time during construction. Where an instrumentman is employed on the work constantly the line is sometimes marked only at the manholes, and the reference marks in such cases may be selected in several ways. The most rapid work is generally possible if two pairs of such marks approximately at right angles to each other can be selected, so that by stretching cords across both, the center point is given. It is not uncommon, however, for the conditions to require setting up the transit over one mark in a pair for locating the center point. An entirely different method is to locate each center point along the sewer line by tie measurements from two or more nails driven into

trees, posts or fences at points clearly described in the notes. If the ring of the steel tape is hung on each nail when the tie lines are measured, one man can give the center line from them without assistance.

The bench marks should be established at frequent intervals and well referenced, for strict compliance with established grades is necessary in sewer construction and leveling can be done more rapidly and accurately where bench marks are conveniently located.

164. Transferring Line and Grade into the Trench.—The terms "line" and "grade" are in constant use in connection with surveys, and although the employment of the words is not strictly in accord with their meanings, it may be accepted as established usage. The "line" of a sewer is the horizontal projection of its axis; the "grade" (contraction of "grade line"), is the profile of the invert, and the "grade" at any point is the elevation of the invert at that point. During construction of a sewer the line and grade must be transferred into the trench, in order that the sewer may be in the proper location and at the correct elevation. This transfer is usually accomplished by the use of batter boards, similar in a general way to those employed in building construction, and described in a subsequent section. In sewer work the batter boards are commonly called "grade boards," and using the definition of "grade" given above, this term is sufficiently accurate to justify its use.

Grade boards (Fig. 95) are set across the trench at intervals, usually of about 25 ft. If the sewer is in an unpaved street, the grade board may be a 2- by 8-in. plank, several feet longer than the top width of the trench, set on edge across the trench with its top 2 or 3 in. above the surface of the ground, with earth tamped about each end until the board is held firmly. In soil stakes may be driven on each side of the trench, to which the grade board is nailed. Where the street is paved and it is undesirable to disturb the pavement more than is absolutely necessary, a 4- by 6-in. or 6- by 6-in. timber may be used as a grade board by weighting the ends with rock or earth until they are immovable.

After a number of grade boards have been set in place, their stationing is accurately determined by measurement, and the sewer line is transferred to them, usually by a transit, and marked by a notch in the edge of the board.

A batten is set vertically on the same side of each board in such a way that one edge of each batten is directly over the center line of the sewer. The same side of every batten must be used over the center line, because the alignment of these battens is checked frequently by sighting along them as the work progresses, in order to be sure that none has been disturbed. The battens may be made of 3- by 1-in. stock dressed on all sides, about 3 ft. long

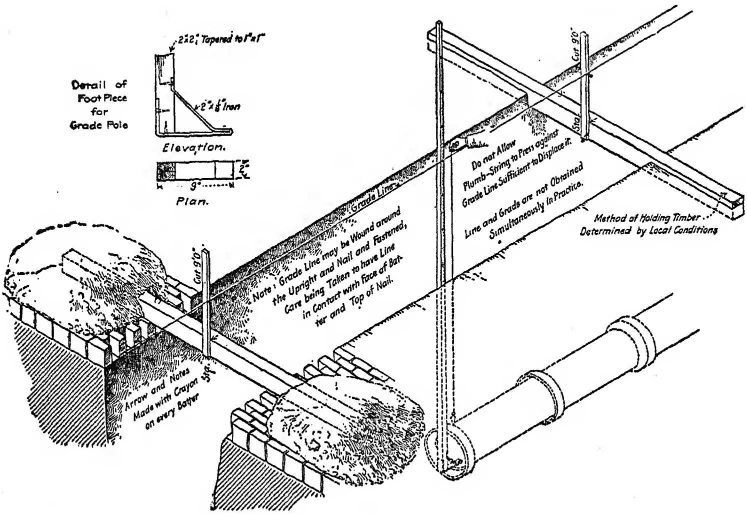


FIG. 95.—Giving line and grade for pipe sewers.

The elevation of the grade line at each grade board is computed, applying the rate of slope to the distance between boards, and adding the rise thus found to the grade elevation at the preceding board. At the end of each section or set of grade boards, the grade elevation should be independently computed from that at the last preceding manhole, as a check of that obtained by adding successive increments of elevation.

After computing the grade elevations, some constant number of feet is added to determine the elevation at each grade board of the cord from which line and grade are to be transferred into the trench. This cord should generally be from one to two feet above the elevation of the ground. Its position is marked upon the center-line edges of the battens by nails. If a stout cord is fastened to a batten, drawn over the nail, stretched to the next batten, wrapped around it just above the nail, and this procedure

continued as far along the trench as desired, the cord will mark a line parallel with the center line of the sewer and a uniform distance above the invert. Furthermore, this line will be so readily seen that it can be checked by sighting along it by the eye, without an instrument.

Grade is transferred from the cord to the invert by a graduated pole with a foot piece, as shown in Fig. 95. This foot piece is placed on the invert of the pipe and the pole held plumb. The pipe is then raised or lowered until it is the right distance below the cord. A plumb line is then held lightly against the cord and the pipe shifted sidewise until its crown is directly below the point of the plumb bob, and the elevation determination is repeated if necessary.

165. Field Notes.—All records made in staking out trenches should be made in standard notebooks, kept thoroughly indexed

56 SEWER GRADES 57

Massachusetts Ave., from Boston St.—West

S.C.S.
E.W.A.
C.R.W.
April 12, 1912

Center Line of Book

Sta.	Size	El. Invert	Rate of Grade	String Above Invert	String Elevation	Rod Readings First	Rod Readings Second		Level Notes B.S.	Level Notes H.I.	Level Notes F.S.	Level Notes El.	Description
						H.I.=68.75		H.I.=68.75		First	Setting	Apr.	12, 1912
4+75	12"	57.07	.008	9.0	66.07	2.68			6.504	68.750		62.246	B.M. #16 rt. cut cor. lower
													stone step, main entr.
5+00	12"	57.27	.008	9.0	66.27	2.48							Trade School
							H.I.=68.21						
+25	12"	57.47	.008	9.0	66.47	2.28	1.74		7.172	74.496	1.426	67.324	T.P. top oper. nut hyd. opp.
													355 Mass. Ave.
+50	12"	57.67	.008	9.0	66.67	2.08	1.54						
													(B.M. #17 top granite foun-
+75	12"	57.87	.008	9.0	66.87	1.88	1.34				2.187	72.309	dation S.W. cor. brick house
													N.E. cor. Mass. Ave. and
6+00	12"	58.07	.008	9.0	67.07	1.68	1.14						Shawmut Ave.
	M.H.	at Sta.	6+00			H.I.=74.50				Check	Level	Apr. 14,	1912
+25	12"	58.24	.00667	9.0	67.24	7.26	0.97		5.961	68.207		62.246	B.M. #16 as above
							H.I.=73.18						
+50	12"	58.41	.00667	9.0	67.41	7.09	5.77		5.864	73.185	0.886	67.321	T.P. as above (0.003 diff.)
+75	12"	58.57	.00667	9.0	67.57	6.93	5.61				0.878	72.307	B.M. #17 as above
7+00	12"	58.74	.00667	9.0	67.74	6.76	5.44				5.49	67.70	Top nail Sta. 7+00 as first
													set. settled 0.040
+25	12"	58.91	.00667	9.0	67.91	6.59	5.27				5.36	67.83	do. +25 do. 0.080
+50	12"	59.07	.00667	9.0	68.07	6.43	5.11				5.21	67.98	do. +50 do. 0.090
+75	12"	59.24	.00667	9.0	68.24	6.26	4.94				4.99	68.20	do. +75 do. 0.040
8+00	12"	59.41	.00667	9.0	68.41	6.09	4.77				4.78	68.41	
+25	12"	59.57	.00667	9.0	68.57	5.93	4.61						

FIG. 96.

during the progress of the work. Construction should never be delayed by imperfect or inadequately recorded work of surveying parties. Fig. 96 gives a form of notes which has proved satisfactory. The notes should give the elevation of the invert for each station where an elevation is required, the slope of the

sewer, its size and all level notes by means of which such grades were determined. All secondary readings used in checking grades should be recorded also. It is of the utmost importance that all work should be checked, preferably by a second independent determination rather than a mere checking of figures when such an independent method can be developed.

In running any levels, as for setting grade boards, the last set-up should be checked on a bench mark. This bench mark should always be a different one from that used at the beginning in order that every group of levels may be tied to at least two bench marks. This method serves not only as a check on the work, but also discloses injury, whether accidental or malicious, to any bench mark. The cut or depth of excavation found by leveling should be checked roughly by comparison with the cut shown upon the profile. On account of the value of records of subsurface conditions, particular attention should be paid to recording the characteristics of the soils, ledge, ground water, pipes, culverts and other underground structures encountered. It is well, also, in recording the date, to enter the weather conditions when the survey is made.

166. Quantity Surveys.—The measurements which must be made for quantity surveys will be governed by the method of payment stated in the contract. If the payment is made on the basis of the length of trench, it is unnecessary to make such detailed measurements as when the payment is computed on the basis of quantity of excavated material. In the latter case, a profile along the center line must be made, showing the ground surface and the profile of the top of all ledge encountered. Care must be taken to make the records of all conditions affecting quantities complete, in order to be able to justify all estimates for payments. It is customary in some engineering offices to prescribe certain limits to which the quantity of excavation shall be paid for, although the contractor may excavate beyond those limits if necessary and, if the structure can be built in a narrower excavation, he is not required to take out material to the full width shown by the "lines of excavation" on the drawings. If the contractor is paid by unit prices, however, it is advantageous that the engineer's notes should show exactly what he did, irrespective of contract agreements regarding "lines of excavation," for it is only by having accurate records of what was accomplished that the actual facts can be reported in case of

need, and such information has real practical value to the engineer. Difficulties often arise on account of insufficient measurements and records, but rarely on account of too many.

In the case of works covering a substantial area, such as pumping stations or treatment works, the surface to be excavated should be cross-sectioned, as should all ledge encountered as the work progresses. All features calling for special payment under the contract should be measured in advance of the work, such as the area of pavement to be removed, the area and quantity of top soil to be removed and kept for future use, and the length of pipe to be taken up for relaying.

167. Location of Branches and Specials.—The length of each size of pipe laid in each section of the work and the character and location of all branches and other specials must be recorded. The recording of the branches by inspectors is often unsatisfactory. It is sometimes the practice, when the engineer is not present all the time the work is in progress, for a pole to be placed at each branch, with its top extending to the surface of the street, so as to enable the engineer to prepare such reference marks or records as he considers best when he next visits the work; but such poles may not remain vertical, or they may be pulled up and lost, or replaced in incorrect locations, so that they are not entirely trustworthy. Some engineers require their inspectors to locate the branches from marks on the sheeting of the trench or by ties to stakes driven on one side of the trench. On account of the many mistakes which arise when branches are located by persons unfamiliar with the importance of the work, it is preferable to leave them uncovered until the engineer can take the necessary measurements. One method of location is by determination of the "station" of the branch or other fitting, by measuring the distance from the preceding grade board. This should be checked independently by the measured distance from the next grade board, and the sum of the two measured distances should be compared with the distance between the two boards. One of the best methods is to measure the distance of each branch, as soon as laid, from the last preceding branch, showing the figures on a sketch which should also indicate on which side of the sewer the branch is situated. The distance of the first branch after passing a manhole is measured from the center of the manhole, and the distance from the last branch before reaching the next manhole, to the center of that manhole is recorded also.

The successive distances are recorded on one side of the line representing the sewer between two successive manholes, and the sum of the distances on the other side. This sum must equal the distance between the manholes, as given by the stationing; and the inspector should be required to make this check and show it in his notes.

It is of the greatest importance that the location of branches be correctly recorded, otherwise much difficulty will be experienced in finding them when building connections are to be laid.

The dimensions of manholes, catch basins, flushing chambers and other structures should be measured as soon as they are completed. The length, size, location and construction of underdrains should be measured and recorded. The dimensions of all sheeting left in place and its location should be recorded, and it is desirable to record the same information relative to sheeting used and removed.

168. Structural Surveying.—The surveying done in connection with the construction of treatment works, pumping stations and similar parts of a sewerage system is performed like the surveying work in building operations. The object is to transfer lines from carefully established bases and elevations from permanent bench marks, and to do this work in such a way as to avoid delaying the workmen. The nature of the construction operations carried on is such that the working gangs can proceed most rapidly if they are provided with means of checking many of the lines and elevations approximately without calling upon a surveyor. Such means are furnished by *batter boards,* generally placed at the main corners and at other places where they will be of service. A batter board is a horizontal board held in place outside the building line far enough to be undisturbed by the work, on posts set vertically and firmly in the ground. On the top edge of this board nails are driven directly on the building lines to which the work is being constructed; at first, in many cases, the line will be that of the footings, and later it will be that of the wall. As a board will often carry a number of nails, each should be marked to indicate the line on which it lies. If a cord is stretched between the nails on two batter boards on opposite sides of a building, it will give the desired line of footing or wall, as the case may be.

The top edges of the batter boards should, if possible, be a whole number of feet above the elevation of the bottom of the structure.

for fractional dimensions in working from batter boards often lead to mistakes. During the progress of the excavation the foreman can be provided with a rod having a crosspiece nailed to it at a distance from one end equal to the elevation of the batter board above bottom grade. By sighting across the batter boards at the top of this crosspiece while the rod is held vertically it is possible to check closely the depth of excavation at any point and thus avoid taking out more material than is necessary.

In doing such surveying, care must be taken to provide one or more base lines which will not be disturbed. These lines should have transit points accurately established on them, which will remain permanent and easily accessible throughout the construction period, and from which the main points on the batter boards can be checked from time to time. In some soils which are much affected by the action of frost and changes of temperature, the main points on the base lines will require markers which will not change their position, for ordinary stakes will not remain in place. Similarly the bench marks must be selected so that their permanence is assured and the surveyor can use them readily as the work proceeds. Care should be taken to have sufficient bench marks to make turning points unnecessary in most of the work, and to permit ready checking of the elevation of the batter boards at frequent intervals as the work proceeds.

Lines and elevations should be given by the surveyor on all forms before concrete is poured, and the correct location of all parts of the structure should be tested by him before the work on them proceeds. It is only in this way that costly errors can be avoided.

INSPECTION

169. Object of Inspection.—The object of inspection is to see that the engineer's plans and specifications are carried out faithfully. Inspectors are assigned to the resident engineer for this purpose, and one of their duties is to safeguard all his reference and bench marks, batter boards and the like, and to check them at frequent intervals as well as they can, so as to detect at once any disturbance of these points which might introduce errors into the work. The inspectors should also see that all transfers of line or grade from the engineer's reference marks by the contractor's forces are made accurately. Workmanship must be inspected constantly in order to ascertain that

all necessary details are given the proper attention. For example, it is essential that pipe inverts be set at the proper grade, the joints made as required, the surplus jointing material removed from the inside of the pipe, water kept away from the joints until they have hardened, concrete mixed properly, the forms built to give a satisfactory surface on the finished work and placed correctly. The eye must be trained to notice any deviation from courses, lines, grades, or standards of workmanship.

170. Excavation and Backfill.—Carelessness in excavation may cause the settlement of adjoining structures, with consequent financial losses. The inspector should, therefore, watch for unnecessary excavation, insufficient bracing of the trench and failure to drive the sheeting sufficiently ahead of the excavation to prevent materials from flowing into the trench. When it seems possible that the excavation may cause settlement or other damage to adjacent buildings, elevations of controlling points, such as steps, corners, window sills and the like, should be taken before the work nearby has begun, and later at intervals as the work progresses. Where the structures are important or damage to them will result in large financial losses it is desirable to have photographs of them taken from time to time, the negatives being dated and signed by the person making them. When the excavation approaches the required elevation the inspector should take precautions to prevent the removal of material from below this elevation except when it is of such a character that it must be replaced by better material.

The backfilling of an excavation must be watched carefully to see that there is no disturbance of the sewer or other structure. Where underdrains are laid it is particularly important to see that the specifications are carried out faithfully and all refilling below the pipe is done thoroughly. As the backfilling progresses the inspector should prevent the dumping of material directly upon pipe or masonry, and make sure that the materials are suitably distributed, spread and compacted. After the trench has been filled to the height of a foot or so above the crown of the sewer, the backfill may sometimes be dumped or scraped into the trench; but where the fall of the material is greater than 5 ft., it should drop first on a timber grillage or other device to absorb the shock.

Sheeting which has been driven below the springing line of a sewer ordinarily should not be removed, for it is difficult to fill

the voids left by such withdrawal. This is true, in even greater degree, of sheeting driven below the bottom of the trench. When sheeting is withdrawn the work should be done a little at a time, and where important the voids should be filled with the aid of a water jet. A piece of ¾-in. pipe, attached to a garden hose, and forced down through the backfilling and moved about while water flows through it, will serve this purpose. The exact method of performing the work will be determined largely according to the character of the ground traversed.

171. Concrete Materials.—The inspector is not usually required to test cement, but he should be watchful of all deliveries to make sure that only cement which has passed the tests is used in the work. Where cement is obtained from a dealer's warehouse this will often require tagging acceptable cement and placing the tagged bags aside for the exclusive use of the sewerage work. Where the cement is delivered to a contractor's storehouse in carload lots, tagging is unnecessary, but no lot of cement which has not passed the tests should be mixed with that which has been approved.

The degree and nature of the inspection of sand, gravel, crushed stone and brick will depend upon the local conditions and specifications for such materials. The inspector must be alert to detect at once any change in the quality of these materials, even if the change is not prejudicial to the work, and to watch for the presence of loam and dirt in aggregates intended for concrete or mortar.

It is even more important to insure that the requisite quantities of cement, water and aggregate are placed in the concrete mixture used. This necessitates a careful checking of quantity as well as of quality of the materials as used.

The deliveries of reinforcing steel should be checked to make sure that they conform with the bill of materials and the cross-sections of the rods should be measured. Absolute adherence to the usual specification that rods shall be free from rust and mill scale is difficult of attainment, and engineers are reluctant to compel a contractor to assume the substantial expense which strict compliance with the requirement entails, in view of the protective effect of the mortar. Serious rusting or pitting is ground for rejection of the steel, but the inspector should get clear instructions from the resident engineer as to what "serious" shall signify on the work in hand, and what may constitute a

reasonable interpretation of the specifications relative to this subject.

172. Cast-iron Pipe.—This should be inspected as it is unloaded from the car and again before it is lowered into the trench. Special attention should be paid to the detection of cracks in the spigot end. The inside and outside are rubbed with a heavy cotton mitten or bunch of dusty waste, which forces enough dust into a crack to make it visible. The presence of a crack can sometimes be detected by the ring of the metal when it is struck with a wooden mallet, while suspended in the unloading slings or on the skids.

173. Vitrified Pipe.—If vitrified pipe is furnished by a manufacturer with a reputation for good mill inspection it is generally considered unnecessary to inspect the pipe at the mill, but because of danger of damage in transportation the pipe should be inspected as it is unloaded from the cars and again before it is placed in the trench. The inspection is made by striking the pipe with a light hammer, and examining it for warping, deformation, defects in the bell and spigot, and poor interior condition. A light tap of the hammer on a pipe set on end will indicate the presence of a crack so small as to be invisible until the surface has been wiped with chalk and the pipe again tapped lightly with the hammer, which will cause the crack to become filled with chalk and visible.

174. Final Inspection.—Notwithstanding that the daily inspection may have been thorough and painstaking, a complete and systematic inspection of the entire work should be made before it is accepted.

The engineer should walk through the entire length of all large sewers, with a powerful lamp equipped with a good reflector, to observe critically their physical condition, cracks, abrasions, leaks, defects or weaknesses of any kind, and the presence of foreign substances, such as lumber, bricks and debris.

All small sewers should be systematically inspected by means of light thrown in either by reflecting sunlight, or by powerful electric lamps with reflectors.

It is well to measure as accurately as practicable the amount of water leaking into the sewer at a time when conditions are favorable to infiltration. At certain seasons of the year there may be little or no infiltration because the sewer lies above the

level of the ground water, while at other seasons it may be under considerable head and the leakage may be significant.

The final inspection of all other structures and equipment must be performed in a manner suited to determine the condition of materials and workmanship used in the particular structure or equipment. Loading tests may be required for strength, while test runs may be necessary to determine whether equipment satisfies specifications.

175. Relationship of Engineer and Inspectors with the Contractor.—The engineer is usually in the position of a judge rather than an advocate, and one of his duties is to interpret the requirements of the contract and specifications with fairness to the two parties to the contract, whose interests may conflict at times. This position is often a difficult one and requires diplomacy, selfcontrol and firmness, to which experience may add an important qualification. The inspector is the engineer's representative on the work during the latter's absence.

A spirit of active and friendly co-operation with the contractor should always be cultivated by members of the engineering corps in the interest of good and rapid construction. Under such conditions mutual respect is usually soon established, and if questions arising in the interpretation of the contract are decided in the light of common sense and fairness, the work should proceed under harmonious and favorable conditions.

Problem

1. Prepare field notes needed for staking out Sta. 13 to Sta. 17 of the sewer shown in Fig. 53. Give elevations at 25-ft. intervals, assuming *H.I.* = 207.00 and cord to be 20.0 ft. above sewer invert below Burr St. and 15.0 for the 8-in. Oak Grove Ave. sewer above Burr St. Note that M. H. at Oak Grove Ave. and Center St. is at sta. 15 + 00, and distances between manholes are given in Table 41, page 172.

CHAPTER VIII

PIPE AND MASONRY SEWERS

PIPE SEWERS

176. Sizes.—Sewer pipes are manufactured and sold as a commercial product in sizes from 4 to 42[1] in. in diameter.

Certain kinds of pipes, such as cast iron, steel, and reinforced concrete, either are manufactured regularly or can be obtained readily in still larger sizes For the purposes of this discussion, however, sewer pipe will be considered as limited by the standard sizes of pipes especially made for sewers with 42 in. as a maximum.

The standard sizes approved by the American Society for Testing Materials (A.S.T.M.) range from 4 to 12 in. by increments of 2 in. and from 12 to 42 in. by increments of 3 in.

177. Requisites.—The principal requirements of any pipe to be used for sewers are strength, durability, imperviousness, smoothness, hardness, uniformity of size and shape, joints of such type that they can be made tight, and low cost for economy of construction.

Obviously, the pipe must have sufficient strength to stand the stresses to which it will be subjected, otherwise it will crack or possibly collapse, and if the sewer is not entirely closed by collapsing, water will leak in or sewage will leak out, and much trouble and expense may result. It must be made of such materials and by such methods that it will be able to withstand indefinitely, or for a long time, the effect of destructive agencies to which it may be subjected; these will vary with locality and conditions of exposure, and may include frost, and corrosive gases or acids or alkalies in the sewage or in the earth in which the pipe may be laid. It must be sufficiently impervious to prevent the admission of a material quantity of ground water, or the escape of sewage into the earth. It must be smooth on its interior surface, in order to avoid excessive frictional resistance and obstruction to flow. It must be hard so as to resist erosion.

[1] Sizes larger than 36 in. in vitrified clay pipe, and 24 in. in plain cement-concrete pipe, cannot be obtained readily at the present time—1929.

It must be uniform in size and shape in order to avoid projections and irregularities at the joints. The joints must be so designed and constructed that they will be and will remain tight. Finally, the cost of the pipe must be such that it can be used with economy when due consideration is given its qualities and to the conditions under which it is to be employed.

178. Kinds of Pipe.—Two kinds of sewer pipe are made commercially—vitrified clay and cement-concrete sewer pipe.

Drain tile is also made of both these materials, and differs from sewer pipe principally in that it is made without sockets (the ends of the lengths being merely butted together), it is not usually salt glazed, and the quality of material and workmanship is somewhat less carefully controlled. Clay tile usually are burned at a lower temperature than sewer pipe, which may result in less blistering, smoother interior surface and less distortion from heat. While drain tile is not suited for sewerage work, it is important to know something of its characteristics, since there are available data from tests upon drain tile which, with some allowances, are applicable to sewer pipes.[1]

Other kinds of pipe are sometimes used for sewers, notably reinforced concrete, which has been employed extensively in sizes for which monolithic concrete and brick sewers also may be considered. Under special circumstances, cast-iron, steel and even wood-stave pipes have been employed.

VITRIFIED CLAY PIPE

179. Manufacture.—Vitrified clay pipe is made from clays or shales, or a mixture of them, which are pulverized in a mill and then mixed with water to form a dough of uniform consistency. This dough or tempered clay is delivered to a press which forms the pipe, operating with a pressure of about 120 lb. per square inch. The clay fills the "mud cylinder" of the press, and is forced out through a die which forms the pipe. The pipes go from the press to the dry room for "seasoning" or drying and shrinking by exposure to warm air, and after this they are burned in a kiln. The heat is carefully regulated, first at about 300°F. for several hours, so as to drive off the remainder of the water without cracking the pipe; then at 1200° to 1400°F. for the oxida-

[1] A noteworthy example is the very detailed series of experiments upon flow of water in drain tile made for the U. S. Bureau of Public Roads, Dept. of Agriculture, by D. L. Yarnell (*Bull.* 854). No such exhaustive experiments upon flow in sewer pipe have ever been made.

tion of organic material in the clay and the gradual escape of the resulting gases; and finally at the vitrifying temperature of 2000 to 2200°F. Under this heat the particles which are subject to melting, most of which are compounds of silicon, flow and surround the more resistant particles, forming a dense, hard, coherent mass. This is the process known as vitrification. During this process, ordinary coarse salt is thrown upon the fires, and under the intense heat the sodium of the salt combines with the silica in the clay to form a glass—the glaze of the pipe. The entire process of burning sewer pipe requires from 2 to 12 days, depending upon the size of the pipe.

180. Shapes and Dimensions.—Originally, each manufacturer of vitrified pipe had his own standards, differing from every other in most particulars except internal diameters. In course of time, however, practice became more nearly standardized and two nearly uniform sets of dimensions came into general use for "standard" and "double-strength" sewer pipe, the latter being made only in 15-in. and larger sizes. Somewhat later the demand for larger and deeper sockets became sufficient to result in the so-called "deep and wide socket," which could be obtained upon either "standard" or "double-strength" pipe. There were then four generally recognized classes of sewer pipe, namely, standard, standard with deep and wide socket, double strength, and double strength with deep and wide socket.

Although dimensions and weights of pipe from various makers were not uniform, the figures given in Table 48 may be taken as a fair representation of the manufacturers' standards.

In an attempt to reduce the number of classes and patterns of pipe in use, the A.S.T.M. adopted (1924) a single class of sewer pipe, in which the thickness of shell is that of the manufacturers' "double-strength" pipe for sizes in which double-strength pipe is made, and in which the dimensions of the socket are intermediate between those of the manufacturers' "standard" and the "deep and wide socket." The dimensions of vitrified clay sewer pipe required by the A.S.T.M. specifications are given in Table 49.

The A.S.T.M. standard of a single class of pipe has not been adopted generally and modification of the specifications is now (1929) under consideration.

There is considerable demand for thinner pipe than the "double-strength" (A.S.T.M.), and the Clay Products Associa-

TABLE 48.—APPROXIMATE DIMENSIONS AND WEIGHTS OF SEWER PIPE (Eastern Clay Products Association, 1925)

Diameter, inches	Single-strength pipe					Single-strength pipe, deep and wide socket					Double-strength pipe conforming, in general, to A.S.T.M. standards					Double-strength pipe, deep and wide socket				
	Thickness of shell, inches	Depth of socket, inches	Annular space, inches	Weight per foot, pounds		Thickness of shell, inches	Depth of socket, inches	Annular space, inches [2]	Weight per foot, pounds		Thickness of shell, inches	Depth of socket, inches	Annular space, inches [1]	Weight per foot, pounds		Thickness of shell, inches	Depth of socket, inches	Annular space, inches	Weight per foot, pounds	
				E. C. P. A.[2]	A. S. P. C.[3]				E. C. P. A.[1]	A. S. P. C.[2]				E. C. P. A.[2]	A. S. P. C.[3]				E. C. P. A.[2]	A. S. P. C.[3]
4	9/16	1 3/4	3/8	7 1/2	9	1/2	2	1/2	10	10	9/16	1 3/4 [1]	3/8	7 1/2						
5	5/8	2	13/32	11	12	5/8	2 1/4	5/8	12	12	5/8 [1]	2 [1]	13/32	11						
6	5/8	2 1/4	13/32	13	15	5/8	2 1/2	5/8	16	16	5/8	2 1/4 [1]	13/32	13						
8	3/4	2 1/2	1/2	20	23	3/4	2 3/4	5/8	25	25	3/4	2 1/2 [1]	1/2	20						
10	7/8	2 1/2	1/2	30	35	7/8	2 3/4	5/8	37	37	7/8	2 1/2	1/2	30						
12	1	2 1/2	1/2	42 1/2	45	1	3	5/8	45	45	1	2 1/2	1/2	42 1/2						
15	1 1/8	2 1/2	1/2	60	60	1 1/8	3	5/8	65	70	1 1/4	2 1/2	1/2	70	75	1 1/4	3	5/8	75	75
18	1 1/4	3	17/32	80	85	1 1/4	3 1/4	5/8	86	90	1 1/2	3	17/32	100	118	1 1/2	3 1/4	5/8	107	118
20	1 3/8	3	17/32	100	100	1 3/8	3 1/2	5/8	105	115	1 5/8 [1]	3 [1]	17/32	125	138	1 2/3	3 1/2	5/8	135	138
21	1 1/2	3	9/16	112	120	1 1/2	3 5/8	5/8	130	130	1 3/4	3	19/32	135	148	1 3/4	3 5/8	5/8	148	148
22	1 5/8	3	19/32	122	130	1 5/8	3 3/4	5/8	145	145	1 7/8 [1]	3 [1]	19/32	155	157	1 5/6	3 3/4	5/8	165	157
24	1 5/8	3	19/32	140	140	1 5/8	4	5/8	150	150	2	3	19/32	175	190	2	4	5/8	190	190
27	2	3 1/2	11/16	195	224	...	...	..	...	...	2 1/4	3 1/2	11/16	215	265	2 1/4	4	3/4	260	
30	2 1/8	3 1/2	13/16	230	252	...	...	..	...	...	2 1/2	3 1/2	13/16	275	290	2 1/2	4	3/4	310	
33	2 1/4	4	13/16	300	310	...	...	..	...	...	2 5/8	4	13/16	340	335	2 5/8	5	1 1/4	335	
36	2 1/2	4	1 1/16	350	350	...	...	..	...	...	2 3/4	4	1 1/16	390	375	2 3/4	5	1 1/4	390	

[1] Not A.S.T.M. standards.
[2] Eastern Clay Products Association, 1925.
[3] American Sewer Pipe Company, 1916.

tion has issued a pamphlet of standards in which the A.S.T.M. dimensions are generally followed for "double-strength" pipe, and dimensions in Table 48 for "single-strength" ("Standard") pipe, but with sockets of the same general shape as the A.S.T.M. pattern. The deep and wide sockets are still furnished by some manufacturers while others have stopped making them.

TABLE 49.—DIMENSIONS OF CLAY SEWER PIPE
(American Society for Testing Materials)

Internal diameter, in.	Laying length, ft.	Inside diameter at mouth of socket, in.[1]	Depth of socket, inches	Minimum taper of socket	Thickness of barrel, in.	Thickness of socket	Approximate weight per foot,[2] lb.
4	2	6	1½	1:20	9/16	The thickness of the socket ¼ in. from its outer end shall not be less than three-fourths of the thickness of the barrel of the pipe.	9
6	2	8¼	2	1:20	⅝		15
8	2, 2½, 3	10¾	2¼	1:20	¾		24
10	2, 2½, 3	13	2½	1:20	⅞		33
12	2, 2½, 3	15¼	2½	1:20	1		45
15	2, 2½, 3	18¾	2½	1:20	1¼		75
18	2, 2½, 3	22¼	3	1:20	1½		105
21	2, 2½, 3	26	3	1:20	1¾		145
24	2, 2½, 3	29½	3	1:20	2		185
27	2½, 3	33¼	3½	1:20	2¼		235
30	2½, 3	37	3½	1:20	2½		300
33	2½, 3	40¼	4	1:20	2⅝		350
36	2½, 3	44	4	1:20	2¾		385
39	2½, 3	47¼	4	1:20	2⅞		
42	2½, 3	51	4	1:20	3		

[1] When pipes are furnished having an increase in thickness over that given, the diameter of socket shall be increased by an amount equal to twice the increase of thickness of barrel.
[2] From "Tentative Standards" issued by Clay Products Association, 1925.

The joint room provided in sockets of the A.S.T.M. design is not sufficient to satisfy those engineers who demand a "deep and wide socket" and it seems unlikely that standardization will be accomplished upon the basis of a single class of pipe. At present some of the makers continue to produce their regular classes of pipe, as above stated, and it may be necessary to modify the A.S.T.M. requirements so as to cover the four classes, although it is to be hoped that standards may be adopted so that the dimensions of any class will be identical throughout the country.

Although the A.S.T.M. specifications extend to pipes 42 in. in diameter, the manufacturers' lists include no sizes larger than 36 in. Under present conditions, and in view of the concrete

backing which would usually be needed for the larger sizes, vitrified pipe sewers larger than 27 or 30 in. in diameter are likely to cost more than monolithic concrete.

181. Physical Properties.—In accordance with the specifications of the A.S.T.M., clay sewer pipe must have the following qualities:

It must withstand internal hydrostatic pressure of 5 lb. per square inch for 5 min., 10 lb. for 10 min. and 15 lb. for 15 min., applied in this sequence, without showing leakage.

Sound pieces of pipe, with all edges broken, and thoroughly dried, must not absorb more than 8 per cent of their weight of water, after boiling for 5 hours.

The average crushing strength per foot of length, with the types of loading shown in detail in the specifications, must not be less than the amounts given in Table 50.

182. Comparison of Test Results with Strength Developed by Pipe as Laid.—A comparison of the requirements as to strength developed under different methods of testing indicates, as has been found to be a fact, that the two- and three-edge

TABLE 50.—PHYSICAL TEST REQUIREMENTS OF CLAY SEWER PIPE

Internal diameter, inches	Average crushing strength, pounds per linear foot	
	Knife-edge and three-edge bearings	Sand bearings
4	1,000	1,430
6	1,000	1,430
8	1,000	1,430
10	1,100	1,570
12	1,200	1,710
15	1,370	1,960
18	1,540	2,200
21	1,810	2,590
24	2,150	3,070
27	2,360	3,370
30	2,580	3,690
33	2,750	3,930
36	3,080	4,400
39	3,300	4,710
42	3,520	5,030

method give substantially equal results, while the sand-bearing method gives results approximately 50 per cent higher.

Professor Anson Marston's experiments have shown that tests with sand bearings indicate with substantial accuracy the strength which will actually be developed by the pipe in the ground, when the "ordinary" method of laying is employed.

The "two-edge bearing" or knife edge requires that the pipe be supported upon a metal bearing 1 in. wide, and the load applied through a similar bearing at the top of the pipe. The "three-edge bearing" involves two strips of wood, with 1 in. clear space between them, as the support for the pipe, and a straight wooden block as the upper bearing through which the load is applied. "Sand bearings" require the use of sand boxes in which the pipe is imbedded in sand, for one-fourth the circumference at both upper and lower bearings.

CEMENT-CONCRETE PIPE

The materials used in cement-concrete pipe are portland cement, suitable aggregates, and water.

183. Manufacture.—Cement-concrete pipes are made in metal molds with a concrete mixture of suitable proportions and consistency, and well compacted by ramming (tamping process) or pressure (packerhead process). In either case the concrete is subjected to great pressure, after which the molds are removed and the pipes are cured from 48 to 72 hours in a room kept warm and damp with low-pressure steam, a water spray, or both. High temperature accelerates the hardening of the concrete. Curing in such a manner as to avoid the evaporation of the water needed for hydration of the cement is of great importance. It is stated that web-like markings on the surface of the pipe are indicative of proper hydration and curing.

Unreinforced, or plain concrete pipe may also be made by the centrifugal process, in which a measured quantity of wet concrete is placed in a mold which is then rotated rapidly about its axis. The centrifugal force packs the concrete solidly against the form, so that the pipe can be removed at once for curing. This process gives a dense, smooth pipe, but is not well adapted to forming a socket joint, and collars have been used for jointing; it can be employed to advantage for pipes of greater length than those called for by the A.S.T.M. specifications. The

centrifugal process has not yet been used much in the United States, except for pipes of reinforced concrete.

184. Dimensions.—The A.S.T.M. standard dimensions for cement-concrete sewer pipe are the same as those for vitrified-clay pipe (Table 49), except that the laying lengths for 4- and 6-in. pipes may be either 2 or 2½ ft., instead of 2 ft. only, as for the vitrified pipes, and that the thickness of barrel for some of the sizes above 21 in. is slightly greater for concrete than for clay pipe.

At the present time (1929) these pipes are being made commercially in 4- to 24-in. sizes. Few, if any, makers are equipped to furnish larger than 24-in. plain concrete pipe.

185. Physical Properties.—In accordance with the specifications of the A.S.T.M., cement-concrete pipe must have the same physical properties as given for vitrified clay pipe.

186. Reinforced-concrete Pipe.—For sizes above 24 in. diameter and up to 108 in., reinforced-concrete pipe may be used. Such pipe may be field-cast, machine-made, or made by the centrifugal process. Field-cast pipe are manufactured on or near the site of the work, and require the use of inner and outer steel forms. Machine-made pipe must be made in a permanent plant where the concrete can be subjected to great pressure, as in the case of plain concrete pipe. This process has been used only for sizes up to 54- or 60-in. In the centrifugal process only an outer form is required and the concrete is compacted by centrifugal force.

The larger sizes of pipes must be compared with monolithic concrete or brick sewers and do not come within the definition of pipe sewers adopted in this chapter, which limits the size to 42-in. diameter.

Socket and spigot joints of the ordinary type may be used for reinforced-concrete pipe. Most of the makers have adopted a joint of the mortise and tenon pattern, in which there is no enlargement of the external diameter, or diminution of the internal diameter, at the joint.

Reinforced-concrete pipe can be made of sufficient strength and tightness to withstand a considerable internal pressure and, therefore, it has been used in some cases for force mains and inverted siphons.

Minimum thicknesses, amounts of steel, and ultimate test loads for reinforced concrete pipe up to 72 in. in diameter have

been tentatively adopted by the American Concrete Institute as given in Table 51.

TABLE 51.—MINIMUM DIMENSIONS AND ULTIMATE STRENGTH REQUIREMENTS OF REINFORCED-CONCRETE SEWER PIPE

(Tentative Standard Specifications, American Concrete Institute, 1925)

Internal diameter of pipe, inches	Minimum dimensions for: Field-cast pipe, Thickness, inches	Field-cast pipe, Minimum area steel, square inch per linear foot	Shop-made pipe, Thickness, inches	Shop-made pipe, Minimum area steel, square inch per linear foot	Ultimate load in pounds per linear foot of pipe (28 days): Three-edge bearing	Sand bearing
24	3	0.065	2½	0.15	3,000	4,500
27	3	0.065[1]	2⅝	0.18	3,300	4,950
30	3½	0.085	2¾	0.21	3,600	5,400
33	3¾	0.105	2¾	0.24	3,900	5,850
36	4	0.125	3	0.36	4,200	6,300
42	4½	0.150	3⅜	0.42	4,700	7,050
48	5	0.210	3¾	0.46	5,100	7,650
54	5½	0.250	4⅛	0.50	5,500	8,250
60	6	0.290	4½	0.54	5,800	8,700
66	6½	0.320	4¾	0.58	6,000	9,000
72	7	0.360	5	0.62	6,200	9,300

NOTE: Sizes below heavy line have reinforcing near both inner and outer faces.

[1] Correctly copied, but probably should be 0.075.

CAST-IRON, STEEL AND WOOD PIPE

187. Cast-iron Pipe.—Where a sewer is under considerable internal pressure, as in force mains and inverted siphons, and where subjected to heavy external pressure as under railroad embankments, cast-iron pipe generally has been used, although in a few instances steel or wood-stave pipes have been employed, and recently the use of reinforced-concrete pipe has become common.

Cast-iron pipe sewers have also been employed in some cases where the quantity of water in the ground was large and where it was particularly desirable to limit the leakage of ground water into sewers as much as possible. The relative ease of making water-tight joints with cast-iron pipe, as compared to sewer pipe, as well as the imperviousness of the iron pipes themselves, has been largely responsible for this use.

Cast iron is also commonly used for the outfall or discharge end of a pipe sewer where it empties into a body of water,

owing to its greater resistance to the action of flowing water, ice, drift, and the like. The submerged outfalls in Boston harbor are illustrations of such use, and similar outfalls in Lake Erie at Cleveland are examples of the use of reinforced-concrete pipe.

188. Steel Pipes.—Under unusual conditions, as for instance where light weight may be of importance, steel pipes have been used for sewers in some cases; but the obstructions caused by rivet heads and joints are objectionable and the metal is subject to comparatively rapid corrosion.

189. Wood pipes have also been used in a few cases, especially for outfall sewers; although the wood does not decay if constantly wet and does not become roughened, nevertheless the bands corrode rapidly, and wood pipes generally have but a short life.

LAYING PIPE SEWERS

190. Method of Laying Pipe.—Great care must be taken in handling sewer pipe to prevent injury to it. Pipes up to 24 in. in diameter are usually taken from the cars by hand, but where larger sizes must be handled in quantities a derrick will probably save time and breakage, although these large pipe can be moved by hand with the aid of skids. If the pipes are placed in storage they should be laid in piles, the sockets of one row alternating with the spigots of the next row, and all pipes securely held by blocking, to prevent a run of the pipe in the lower rows. If pipe has to be carried through the winter, which is undesirable, it should be supported so that it cannot become frozen to the ground and so that water will not collect in it. The pipes should not be taken from the yard to the street, as a rule, until a day or so before they are needed, owing to the greater danger of breakage at the site of the work.

Pipes are generally lowered into the trench by hand, but when they are 24 in. in diameter or larger a three-leg derrick which can be set up across the trench is useful in handling them. Where a sling is desirable in lowering them what is known as a "hook rope" is often used. This is a rope with a hook spliced in one end and a chain and hook spliced in the other end. The chain is passed through the length of pipe and the hook fastened over the chain at the spigot end of the pipe, thus forming a loop in which the pipe hangs as it is lowered into the trench.

After being lowered into the trench the pipe is usually shoved into place by the pipe layer, his helper lifting the spigot end by means of a rope, strap, or wire passed under the pipe. If the pipe is so large as to be hard to move in this way a lifting hook or pipe buggy, Fig. 97, may be used.

If, when the pipe is in place and tested for line and grade, it is found to be slightly too high, it is sometimes possible to force it down to grade by lightly tapping it on the trench bottom, but otherwise it must be lifted and just enough earth scraped out to lower it to the required elevation, and at the same time to give it a uniform and firm support. If the pipe is found to be too low when tested it must be raised and earth carefully spread below it so as to bring it to grade when it is again lowered. The bottom

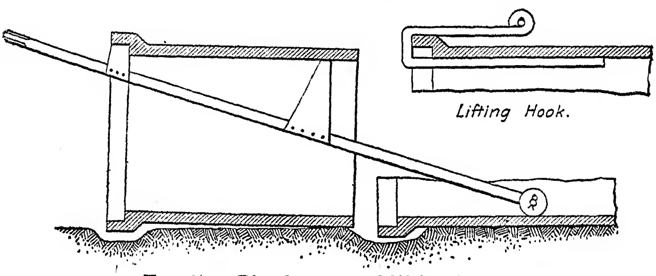


FIG. 97.—Pipe buggy and lifting hook.

of the trench in earth should be shaped as nearly as possible to fit the lower quadrant of the pipe, with a hole under the socket to enable the joint to be made. If the trench is in ledge the rock is generally taken out 6 in. below the invert of the sewer and the space refilled with good earth or gravel.

In most sewer pipe laying three or four men constitute a working gang. One handles the pipe on the bank and lowers it into the trench, another receives it and places it in position, another makes the joint and, with the fourth, carefully tamps the earth under and around the sides and fills the socket hole completely. The filling and tamping until the backfill is a foot above the pipe must be done carefully to avoid disturbing or cracking the pipe. It must proceed equally on both sides of the pipe and the tamper should have a face not exceeding 5 by 1½ in. and should not weigh over 7 lb. For work about the sides of the pipe a tamper of even smaller size is useful; it has a face about 4 in. long and ½ in. wide and a handle of ⅝ in. iron set at an angle so that the filling under the pipe can be reached easily.

191. Effect of Method of Laying upon Strength of Pipe in the Ground.—The relation between the strength of pipe as developed by laboratory test and that which will be developed when the pipe is laid in the ground has been found by Marston[1] to depend upon the character of the bearing between the pipe and the supporting earth. This means not only the earth upon which the pipe is laid (bottom of trench) but that on either side (walls of trench).

If the trench is excavated with a flat bottom, upon which the pipe is laid, the resulting condition will approximate that of the two-edge or three-edge laboratory test. The pipe so laid cannot be expected to develop more than 80 per cent of its strength as shown by sand-bearing tests. It should never be allowed and is, therefore, called the "impermissible" pipe laying method.

"Ordinary" pipe laying requires that the bottom of the trench be shaped to fit the pipe for a bearing arc of 60 to 90 deg. The backfill is placed loosely over and around the pipe, which may be expected to develop just about its test strength as shown by sand-bearing tests.

"First class" pipe laying requires the pipe to be especially well bedded for at least 90 deg. of its circumference, and the backfilling well tamped up to the level of the top of the pipe, thus ensuring a firm bearing between the sides of the pipe and the walls of the trench. It is possible to develop about 120 per cent of the strength shown in laboratory tests with sand bearings, in pipe laid in this way.

192. Platforms and Cradles.—The ideal material in which to lay a pipe sewer is that which can be shaped to fit the lower half of the pipe and which may be refilled firmly and solidly about the upper half of the pipe. Trenches in such material do not require sheeting to a depth below the center of the pipe, The only material which fulfills these requirements is a firm sand such as is only occasionally encountered. Even in this material it is rarely possible to realize the desired result as it is impracticable to cut a trench exactly to the shape of the pipe. which is seldom of true geometrical shape.

Where the trench is deep, particularly when the size of the sewer is large—15 in. or over—and especially when the soil is yielding, additional support from a concrete cradle should be provided. By the use of a concrete cradle under the pipe and

[1] Second Progress Report on Culvert Pipe Investigations, 1915–1921.

extending to a height of about one-third the diameter, a strength from 80 to 100 per cent greater than the test strength of the pipe, with sand bearing, can be developed, as indicated by tests upon vitrified pipes up to 24 in. in diameter. Examples of concrete cradles are shown in Fig. 98.

In quicksand and similar material it may be necessary to carry down the bottom of the trench by special expedients, and in carrying them out it is important to do the work rapidly and reduce the walking about on the bottom of the trench to a minimum, for the more such material is stirred up the more difficult it is to control. Usually it is necessary to provide for

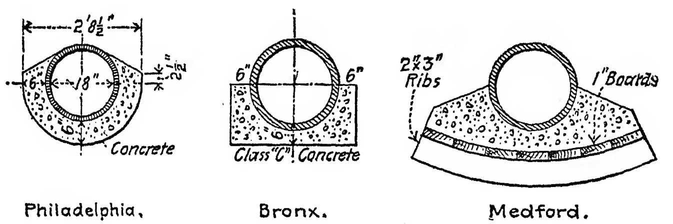


FIG. 98.—Types of cradles.

draining the ground by the use of an underdrain or by a layer of gravel through which the water may be led to a pump or permanent outlet. Under some conditions a good foundation may be provided by excavation for one or two feet below the sewer grade with a refill of gravel or crushed stone. If the natural ground is very fine, particularly if it contains clay, a layer of coarse sand below the gravel may be necessary, to prevent this material from entering the interstices in the gravel. In all soft ground construction it is particularly important to provide support for the pipe for its entire length. For this reason the support of the sewer by the use of blocks placed under the ends of succeeding pipes should not be allowed.

In very wet trenches in sand, where the pressure of the water causes the material to flow in or rise in the trench, timber platforms braced to the trench sheeting are occasionally used, to hold the bottom down. The construction of the platform may be commenced by first setting two sills of 6- by 6-in. timbers or larger, or planks laid on edge longitudinally with the trench. When these are placed at the proper depth they may be held in place by bracing from the sheeting and cross planking may be

placed upon them extending to the sides of the trench. This planking should be spiked to the sills and it may be necessary also to brace the ends of the planks to the sheeting.

When the platform is in place construction of the sewer may proceed. If the material excavated is a coarse clean sand it may be used as a filling between the platform and the sewer. If the natural soil is of a clayey nature it should not be used below the sewer, and a suitable material such as sand or gravel or possibly concrete, should be substituted.

Whenever the depth of backfill or the size of sewer necessitates the use of a concrete cradle, this may be constructed upon the platform built as described above or as shown at the right in Fig. 98.

193. Cement Joints.—Most pipe sewers are laid with cement joints. The cement may be used neat or with sand in a 1:1 or 1:2 mortar. The more cement there is, the more easily can the joint be filled, but the more likely the joint will be to crack. More time is required for a mortar joint to harden than one of neat cement. A 1:2 mixture is most commonly used, except when rapid setting is desired.

It is usual to begin a joint by placing a gasket formed of a piece of jute or oakum soaked in cement grout around the end of the spigot of the pipe to be laid. Its purpose is to keep the pipe centered in the socket of the pipe already in place and prevent the squeezing out of the mortar in the bottom of the joint by the weight of the pipe. If no gasket is used, mortar is spread in the lower part of the socket of the pipe already in place in order to hold the new pipe in place, but this mortar may be forced out to some extent and joints made in this way are not so good as those made with a gasket.

After the pipe is in position, and water is removed from the bell hole so that the joint can be made "in the dry," a man with a trowel and rubber glove fills the joint carefully with a dry mortar, working it into place with a wooden calking tool. The mortar is usually left with a slope of about 45 deg. from the outer edge of the bell to the pipe with which the joint is made. This is termed "overfilling" the joint, and is generally believed to give a tighter joint than finishing off the mortar flush with the end of the bell.

After a joint has been made it is desirable to wrap about it a piece of cheese cloth, securely tied at the top, to hold the mortar in place, for there is a tendency for the mortar to sag away, particularly if it is rather wet, and thus open the joint

somewhat. The use of the cheese cloth also permits the use of a rather moist mortar with which a better joint can be made than with a dry mortar.

Except where a gasket is used there is a tendency for mortar to work through the joint into the interior of the pipe, where it will harden into an obstruction to the flow of sewage if not removed. It is therefore desirable to scrape the inner surface of a pipe sewer as it is laid, for which purpose a "go-devil" can be used. This consists of two wooden disks slightly smaller than the bore of the pipe. Each disk is made of two thin pieces of wood clamped together with a piece of cloth-insertion rubber packing between them, cut to fit tightly in the bore of the pipe. The disks are fastened together about 2 ft. apart by three ¼-in. iron rods. A ¾-in. manila rope passing through and fastened to the disks is threaded through the pipe as laid, after which the go-devil is pulled along, effectually cleaning the sewer pipe. A poor substitute for this is the swab or "half moon" made from a piece of board cut to the curve of the interior of the pipe but not filling it. This board, fastened to a stick a little longer than one pipe, is drawn along first on top and then on the bottom, to clean the pipe.

194. Sulphur and Sand Joints.—A composition of sulphur, tar and sand has been used for many years in England in making the Stanford pipe joint, which has found little or no favor in the United States.

Joints of sulphur and sand have been used to some extent in the United States for many years, however, particularly in connecting several lengths of pipe on the ground before lowering them into the trench. The purpose of the sand is to keep the jointing material at a constant volume while it is cooling and prevent the shrinkage which occurs if little or no sand is used with the sulphur. The sand must be very fine and without a gritty feeling when rubbed between the fingers. Equal parts of sulphur and sand are generally used. The temperature of the material must not be raised above that necessary to keep it liquid, for too high a temperature will change the liquid to a plastic mass, which cannot be used until it cools again to the correct degree. The material is heated and poured like lead in jointing water mains, an asbestos jointer being useful in retaining it in the joint; but the funnel[1] should be higher, at least 3 in. high and ½ in. diameter at the throat. The joint is absolutely

[1] Generally called a "gate." in accordance with casting practice.

rigid when finished, and some engineers prefer a more plastic joint.

195. Asphalt and Tar Joints.—These are generally made with asphaltic preparations of the general character used in pavement work for joint fillers. The trench and socket must be dry before making the joint. The joint is calked with oakum and then the hot filler is poured into it as lead is poured into the joint of a cast-iron water main. When the jointer is removed the joint is overfilled with cement mortar. The material has a low melting point, which prevents its successful use in places where large quantities of hot water or steam find their way into the sewer, as is the case in some large cities.

In the case of concrete pipe, the asphalt will adhere to the pipe more firmly if the inside of the socket and outside of the spigot are painted with a suitable asphaltic priming paint before the joint is made. It is possible that this might be advantageous also for joints in vitrified pipe.

In a large mileage of sewers in very wet soil where it was particularly desirable to prevent the infiltration of ground water into the sewers through the joints, Alexander Potter used the following method: The inside of the bell and the outside of the spigot were swabbed with tar and a gasket of jute soaked in tar was forced into the joint. Into a bucket full of portland cement tar was slowly poured and kneaded by hand until the mixture was of the consistency of rather stiff dough. This was then rolled on a board into a roll of proper size to be forced into the joint, where it was calked until the space was filled to within ½ in. of the end of the bell, after which an overfilled cement finish was given to the work. In this work the spigot had to be supported while the joint was hardening or the weight of the pipe would cause the filling material to exude from the joint, and some difficulty was experienced at first in conducting the work so as to avoid such troubles.

196. Other Plastic Joints.—Pipe joints have been made of several other preparations consisting of bitumen and an inert filler mixed approximately in equal proportions. Such compounds used at the present time include "G-K" compound, "Eastern Clay Goods Sewer Seal," "Puro-Seal" and "Jointite."

Joints made with plastic compounds have the advantages of better adherence to the smooth surface of the pipe; watertightness; resistance to acids and alkalies; sufficient plasticity to

permit of slight movement of the pipe without injury; and the saving of pumping since water may be allowed to rise upon the joint without delay. The poured plastic compounds also have advantage over asphalt and tar, by reason of their higher softening temperature, so that the joint is not as readily affected by hot water and steam.

The compound is shipped in containers and should be broken into small pieces and melted by heating to such temperature that it may be poured "like water."

In jointing the pipe, the joint is first thoroughly calked with dry jute (untarred), and a joint runner is placed around the pipe against the bell and clamped at or near the top, acting as a stop to prevent leakage from the joint space when the compound is poured. The melted compound is then poured through the opening. Especial care should be used to entirely fill the joint.

To facilitate the work, two sections of pipe are sometimes jointed on the bank, the pipe being set vertically while making the joint. If this is done, only one joint need be made in the trench for each two lengths of pipe. In handling the sections jointed on the bank, care should be taken to avoid injury to the joints.

MASONRY SEWERS

For the purpose of this discussion, a masonry sewer is considered to be one built in place in the trench, whether the material be brick, vitrified segment block, monolithic concrete, or a combination of materials.

197. Types of Cross-section.—The majority of the masonry sewers constructed in this country have been of circular cross-section, although in some old systems many sewers constructed with an oval or egg-shaped section are to be found. Since about 1900 a number of other sections have come into use and some of them have found quite general favor. In the following paragraphs, the principal types are described and some of their chief advantages and disadvantages are discussed. The names given to the sections are usually descriptive of the form of the arch or upper part of the section, but are sometimes inaccurate.

198. Circular Section.—The circular section encloses a given area with the least perimeter and on that account affords the greatest velocity when half full or full. Under ordinary conditions, circular sections are economical in the amount of masonry

required, although in flat-bottomed trenches or under conditions requiring special foundations, such as piles or timber platforms, additional masonry is required to support the arch. In the combined system, where the dry-weather flow of sewage is very small in comparison to the storm-water flow, the velocity for the low flows is comparatively small in the circular section and on that account this section may not be as advantageous, theoretically, as the egg-shaped section.

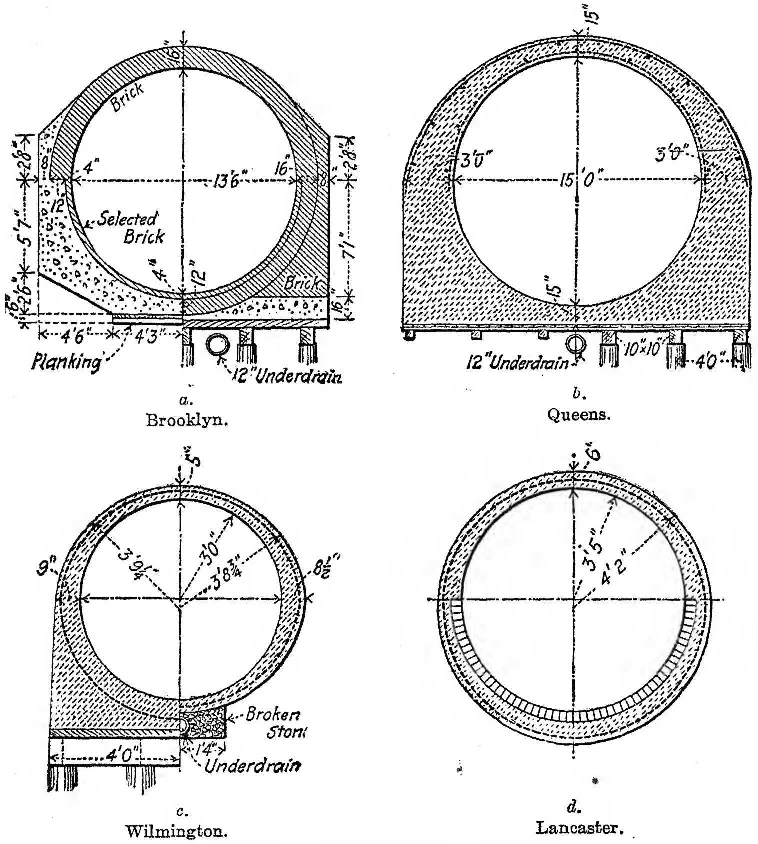


FIG. 99.—Typical circular sections.

For sewers under 5 ft. in size the circular form is usually employed in preference to other types. This type is shown in Fig. 99.

199. Egg-shaped Section.—In combined sewers where the dry-weather flow of sewage is small compared with the capacity of the sewer required for storm water, or in separate sewers for

a district where the present population is but a small proportion of the ultimate development, the ideal sewer section is one in which the hydraulic radius remains constant as the depth of flow decreases. It is impracticable to obtain the ideal, but the egg-shaped or oval section, theoretically, comes nearer to it than any other thus far devised. It is illustrated by Fig. 100.

The advantage of the egg-shaped sewer is that for small flows the depth is greater and the velocity somewhat higher than in a circular sewer of equivalent capacity. The depth of flow in the egg-shaped sewer is always greater than in the circular sewer for equal quantities, and for the small flows this increase in depth produces better flotation for the solid matter and consequently better actual velocity than if the floating solids should become stranded, thus causing obstructions to the flow.

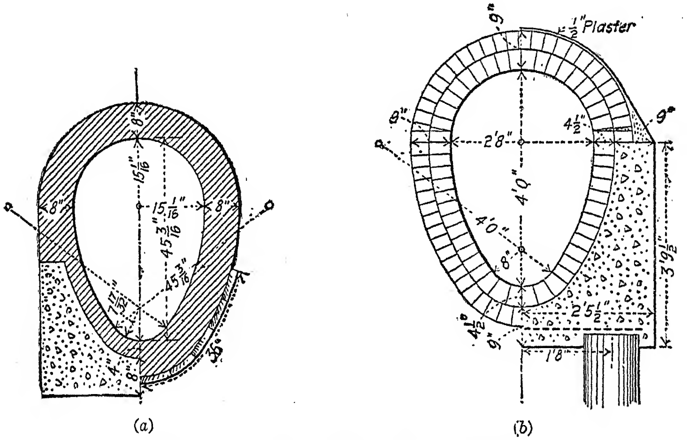
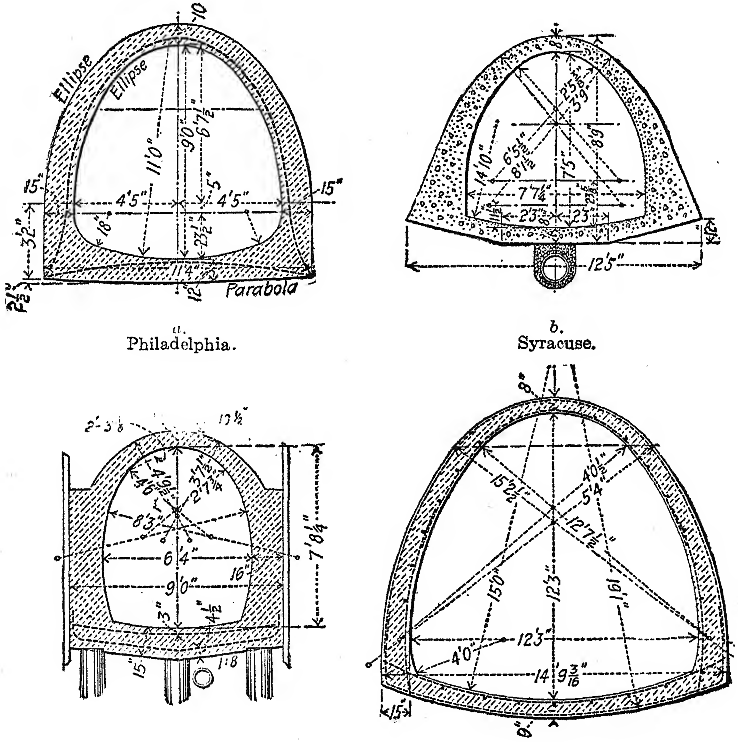


FIG. 100.—Typical egg-shaped sections.

The egg-shaped section has the disadvantages of being less stable, more likely to crack, requiring more masonry, and being more difficult to construct than the circular section. In very stiff soil or in rock, it is sometimes possible to excavate the bottom of the trench to conform to the shape of the invert of the sewer, but, in general, in yielding soil or where foundations are poor, requiring piles or timber platforms, the egg-shaped section requires considerable masonry backing below the haunches to support the arch, even more than in the case of the circular sewer. For this reason, the egg-shaped section will be found in most cases more expensive than the circular type and, in the

larger sizes, far more expensive than some of the other types which are discussed further on.

200. Semi-elliptical Sections.—The arch of this section, Fig. 101, Table 52, is either a true semi-ellipse or is made up of three circular arcs approximating a semi-ellipse. As the center of gravity of the wetted area is lower than in a circular sewer, the normal flow line will be much lower, which may be of some



a. Philadelphia.

b. Syracuse.

c. Boston.

d. Louisville.

FIG. 101.—Typical semi-elliptical sections.

advantage in locating lateral connections lower or in raising the invert of the main sewer. This, of course, contemplates the possible operation of the lateral sewers under a head at times when the main sewer is running full. Although this is of material advantage where the allowable difference in water level is small, it should be avoided generally.

The chief advantage of this type of sewer is that the shape of the arch more nearly coincides with the line of resistance under

actual working conditions than is the case with other sections. Because of this, the arch section can be made relatively thin and still keep the stresses in the masonry within allowable limits. The section is dependent for stability on the lateral pressure of the earth to only a small extent. The fact that the arch is of thin section and goes nearly to the invert makes it more necessary to design and construct the invert so as to distribute the pressure over a sufficiently large area. This section depends to a larger extent upon the stability of the invert than do most of the other sections. Where the semi-elliptical section is constructed in compressible earth and the structure is built monolithic, with reinforcing bars running continuously from the center of the invert to the crown of the sewer, there will be a large bending moment at the center of the invert. Under such foundation conditions, the invert should be made as thick as the arch at the springing line and should be heavily reinforced to withstand the stresses. Unless this is done, cracks are likely to occur in the center of the invert.

Table 52.—Dimensions of Authors' Semi-elliptical Sewer Section (See Fig. 101)

1	2	3	4	5	6	7	8	9	10	11	12
			Thickness of concrete		Interior radii			Exterior radii			
Inside vertical diameter	Area of waterway	Hydraulic mean radius	Crown	Center of invert and spring line	Crown intrados and side wall	Side intrados	Invert and side extrados	Crown extrados	Invert	Area of concrete	Quantity of concrete cu. yd. per lin. ft.
ft. in.	sq. ft.	ft.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	sq. ft.	lin. ft.
6 0	28.2	1.442	0 6	0 9	2 0	6 3	7 6	2 6	8 3	14.12	0.523
6 6	33.1	1.562	0 6½	0 9¾	2 2	6 9¼	8 1½	2 8½	8 11¼	16.58	0.614
7 0	38.4	1.683	0 7	0 10½	2 4	7 3½	8 9	2 11	9 7½	19.21	0.712
7 6	44.05	1.803	0 7½	0 11¼	2 6	7 9¾	9 4½	3 1½	10 3¾	22.08	0.817
8 0	50.1	1.923	0 8	1 0	2 8	8 4	10 0	3 4	11 0	25.10	0.930
8 6	56.6	2.043	0 8½	1 0¾	2 10	8 10¼	10 7½	3 6½	11 8¼	28.35	1.054
9 0	63.4	2.163	0 9	1 1½	3 0	9 4½	11 3	3 9	12 4½	31.80	1.177
9 6	70.7	2.284	0 9½	1 2¼	3 2	9 10¾	11 10½	3 11½	13 0¼	35.41	1.311
10 0	78.3	2.404	0 10	1 3	3 4	10 5	12 6	4 2	13 9	39.24	1.453
10 6	86.3	2.525	0 10½	1 3¾	3 6	10 11¼	13 1½	4 4½	14 5¼	43.26	1.602
11 0	94.75	2.646	0 11	1 4½	3 8	11 5½	13 9	4 7	15 1½	47.48	1.757
11 6	103.5	2.764	0 11½	1 5¼	3 10	11 11¾	14 4½	4 9½	15 9¾	51.89	1.921
12 0	112.75	2.884	1 0	1 6	4 0	12 6	15 0	5 0	16 6	56.51	2.092
12 6	122.4	3.005	1 0½	1 6¾	4 2	13 0¼	15 7½	5 2½	17 2¼	61.31	2.270
13 0	132.4	3.125	1 1	1 7½	4 4	13 6½	16 3	5 5	17 10½	66.32	2.456
13 6	142.7	3.245	1 1½	1 8¼	4 6	14 0¾	16 10½	5 7½	18 6¾	71.51	2.649
14 0	153.5	3.365	1 2	1 9	4 8	14 7	17 6	5 10	19 3	76.91	2.849

Area of waterway $=0.7831D^2$. Area of concrete section $=0.3924D^2$.

As in the horseshoe type, the invert of the semi-elliptical section readily conforms to the bottom of the trench excavation

and for that reason the quantity of masonry below the springing line is not excessive.

This section is not as advantageous for low flows as the circular, because of the wide and shallow invert in which there is a low velocity. However, for sewers where the quantity to be carried is not subject to wide variations and the normal flow is as much as one-third of the total capacity of the sewer, this disadvantage may be neglected. The hydraulic properties of the semi-elliptical section are good in general, which, with the desirable structural features, make this type one of the best for sewers over 6 ft. in width.

201. Other Sections.—The *horseshoe section,* Fig. 102 has been widely used. Above the springing line it has a semi-circular arch, while the side walls below the springing line are vertical

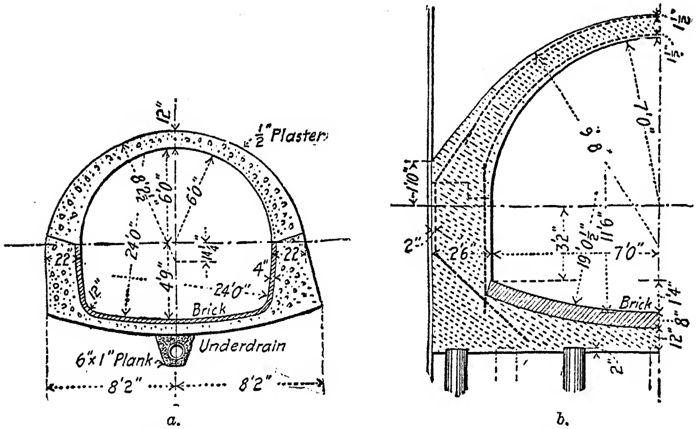
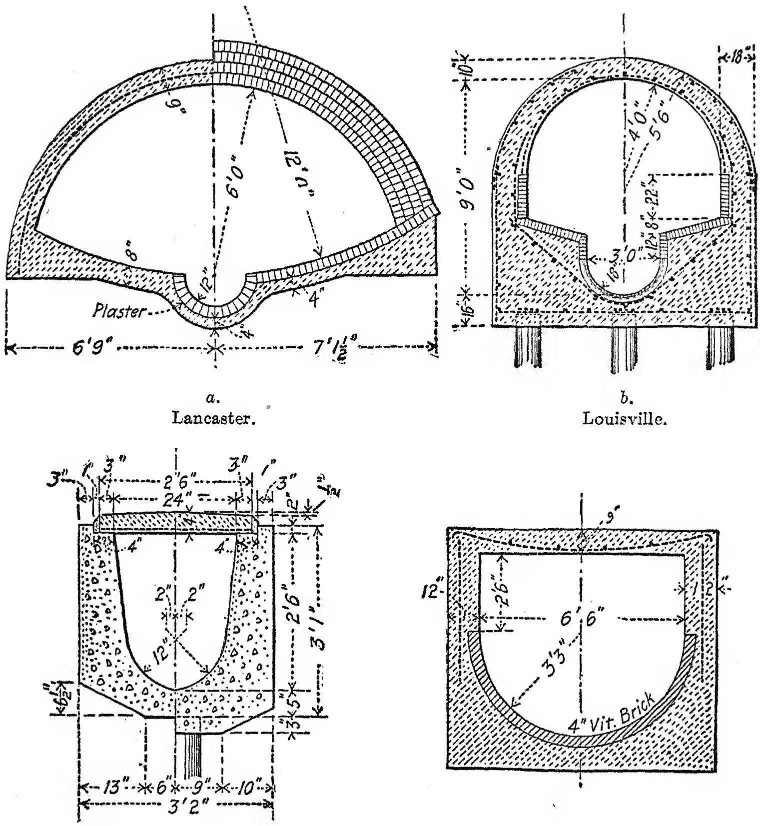


FIG. 102.—Typical horseshoe sections.

or incline inward, sometimes with a plane and sometimes with a curved surface. The invert varies in section from a horizontal line to a circular or parabolic arc, or other design calculated to concentrate the low flows near the center of the invert. The invert conforms to the trench bottom, saving masonry. For a given width, this section will have less height than the equivalent circular sewer. Its main disadvantage is that it depends upon the ability of the back-filling to resist the lateral thrust of the arch transmitted to the side walls near the springing line.

The *parabolic or delta section* has a somewhat larger carrying capacity than a circular section of the same height. It is both economical and strong, the normal flow line is lower than in the circular section, and the sloping invert is well adapted for low



a. Lancaster.

b. Louisville.

c. Syracuse.

d. Richmond.

FIG. 103.—Cunette and U-shaped sewer sections.

flows. It requires a wider trench than the semi-elliptical section for equal capacity and height.

The *U-shaped section,* Fig. 103, has fairly good hydraulic properties until it becomes filled, when the width of the roof causes a large increase in the wetted perimeter. The invert adapts it for low flows and is easily constructed. It requires considerable masonry in proportion to its area, and its main

field is for sewers about 3 ft. wide and considerably over 3 ft. in depth.

The *rectangular section*, Fig. 104, has been steadily increasing in favor because of its excellent hydraulic properties until it is

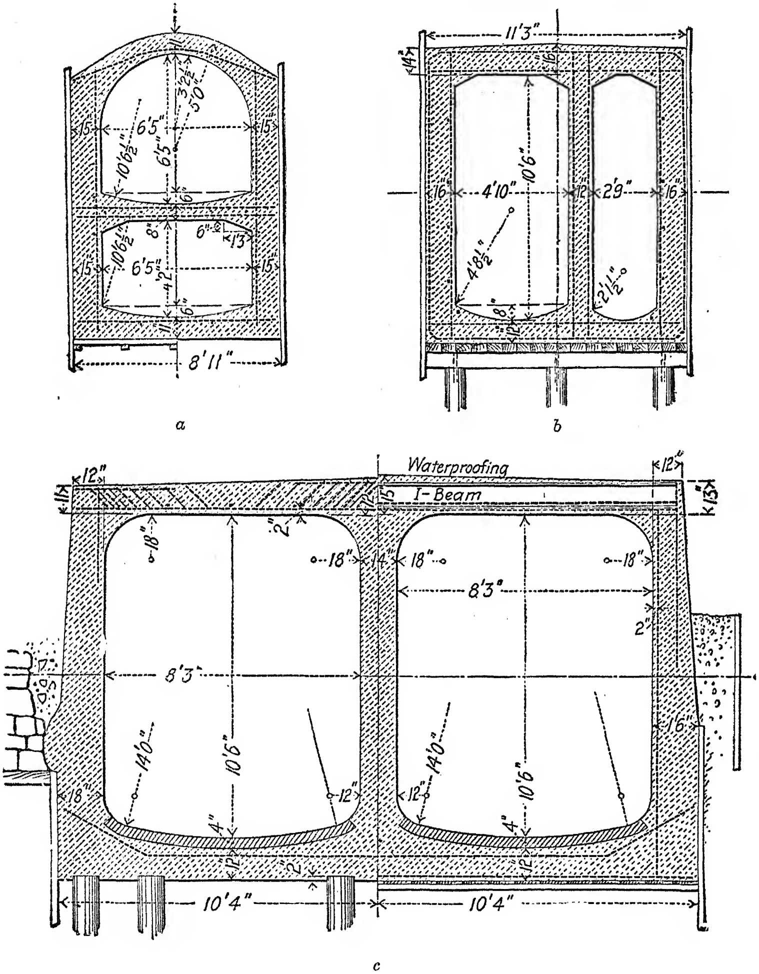
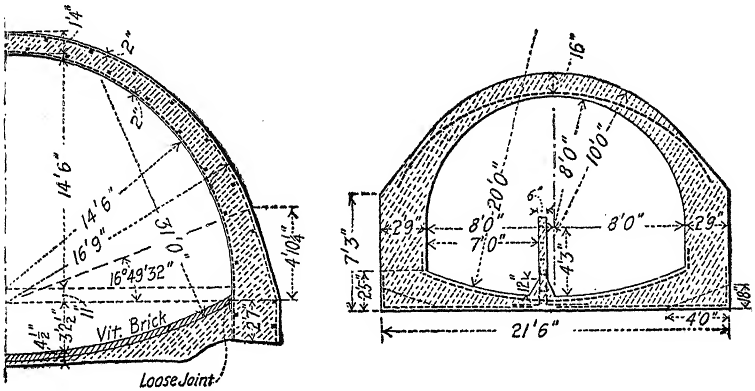


Fig. 104.—Typical rectangular sections. (Boston.)

filled, simplified form work, economy of masonry and space in the trench, and ease of construction. The V-shaped invert is frequently used with the rectangular section on account of its suitability for low flows.

The *semi-circular* section, Fig. 105, was formerly used for large sewers on low land, where the natural ground surface was below the top of the sewer. The rectangular section has largely



a.
St. Louis.

b.
Chicago.

FIG. 105.—Typical semi-circular sections.

displaced it as being less expensive and possessing better hydraulic properties.

SELECTION OF TYPE OF SEWER

The selection of the type of sewer depends upon a number of conditions, all of which must be considered carefully and balanced in the choice of the best type to build. In general, that sewer is the best which will have the requisite stability for the least cost per linear foot, will be easy to maintain in operation and to withstand the external and internal forces. In the following paragraphs a number of the principal items to be considered are enumerated.

202. Hydraulic Properties.—Diagrams of the principal hydraulic elements of a number of sewer sections, based upon Kutter's formula, have been given in Chap. IV (Figs. 38 to 42, pp. 131 to 133).

Theoretically, the best cross-section for a sewer, from the standpoint of hydraulics, with a given slope, S, and carrying a uniform quantity of water per unit of time, is the semi-circle for an open channel and circle for a closed channel, both entirely full; because the hydraulic mean radius, R, has a greater value for these

sections than for any other of equal area, and consequently, on the basis of the accepted formulas, the velocity of flow is greater.

This theoretical advantage is partly offset by the fact that the flow in sewers is not uniform but is constantly changing in depth, and, therefore, the minimum velocity is an important consideration. The circular section is not as advantageous as the egg shape for low flows.

In some cases, a small semicircular channel has been constructed in a V-shaped invert to carry the minimum flow. Assuming a certain minimum velocity is to be maintained with a given minimum quantity of sewage, the diameter of the small semicircular channel required for this flow can readily be computed. The V-shaped invert with circular arc at the junction has been used to advantage with the rectangular sewer section and also with some of the other types.

Where the normal flow is equal to one-third or more of the maximum flow, the circular type is the best for velocity and carrying capacity, but there are other considerations which usually affect the form of the sewer and may dictate some other type.

In comparing one section with another, it is important to study the relation between the depth of flow and the corresponding velocity and discharge. The diagrams in Chap. IV give, for each of the principal types of conduits, the ratio of each of the three hydraulic elements, area, mean velocity and discharge of the filled segment to that of the entire section, corresponding to any ratio of depth of flow to vertical diameter, according to Kutter's formula, but it must be remembered that the applicability of this or any other formula to conditions of flow when the sewer is more than half full and less than full, is at least open to question.

203. Construction and Available Space.—The method of constructing a sewer, whether in open cut or in tunnel, may have an important influence on the selection of the type. In tunnel work especially, it is desirable to have a section which will utilize to the best advantage all of the space inside the tunnel bracing. In earth tunnels, where the common form of timbering is used, the semi-elliptical sections conform readily to the available space. In rock tunnels, the circular or horseshoe sections are likely to be more advantageous. If the sewer is built in open

cut, the form of section will be influenced by its ability to carry the earth loads.

Where the excavation is in rock or firm soil, it is often possible to shape the bottom of the trench to conform to the shape of the invert of the sewer and thereby save considerable thickness of masonry in such types as the circular or egg-shaped sections. If the excavation is in soft material, where the bottom of the trench must necessarily be flat, or if the sewer is to be built on piles or a timber platform, considerable additional masonry will be required for the circular or egg-shaped sewers.

The amount of space available for a sewer may be exceedingly limited. Sometimes the head room is limited because of the proximity of the surface of the street; often the side room is limited because of adjacent structures; and again the available depth may be limited on account of tide water or other conditions which control the allowable depth to the hydraulic grade line. The rectangular section has proved one of the most useful for such conditions, although the horseshoe section, with horizontal and vertical diameters adjusted to meet the conditions, has been used frequently. In a few cases, the full elliptical section also has been used in restricted places. Where the depth to the hydraulic grade line is limited, it is desirable to use a sewer section which will carry the maximum and minimum flows with the least variation in depth of flow. The parabolic, semi-elliptical, and rectangular sections are especially suitable for this purpose, as the center of gravity of the wetted area is comparatively low in contrast to the circular section. The semi-circular section also has proved useful in this connection, although the rectangular section is being used in some of the more recent work of this character.

204. Cost of Excavation and Materials.—The cost of excavation required by one type as compared with another should be considered carefully, for if the excavation is in a deep trench, it will probably be cheapest to use a narrow and deep section and thereby save considerable width of excavation, even though the depth of excavation be slightly increased. This will be especially true in a rock trench, where it may be found of advantage to use a narrow rectangular section having a height 1½ to 2 times the width. For a sewer built in very shallow cut, or practically on the surface of the ground, a wider section will be advantageous, because little additional cost is incurred by increasing width,

whereas greater depth may increase materially the cost of excavation. Furthermore, the cost of an embankment over a wide section will generally be less, because of reduced height. The parabolic or delta section is especially useful for crossing low lands where the sewer is largely out of the ground and must be covered by an embankment. The semi-circular section has also been used for this purpose, but has been superseded more recently by the rectangular section, having a width about $1\frac{1}{2}$ times its height.

In general, from the structural viewpoint, a self-supporting sewer is more desirable than one which depends partly on the passive resistance of the earth backfilling for its stability. When comparing the relative costs of sewers of several types, care should be taken to see that the masonry sections are structurally comparable. For example, it is obviously unfair to compare a monolithic concrete sewer with a two-ring brick sewer, unless account is also taken of their relative stabilities.

The object in designing a sewer section should be to obtain one in which the quantity of masonry and other materials is a minimum consistent with the requisite stability, hydraulic properties, and other considerations.

205. Stability.—The structure must be designed to carry the load of earth or backfill above it as well as any superimposed load. The circular arch is not as strong as either the Gothic, the parabolic, or the semi-elliptical arch. The semi-circular arch depends for stability to a great extent upon the lateral pressure of the sides of the trench, and also to a certain extent on the lateral resistance or passive pressure of the earth backfilling although this can be obviated by increasing the thickness of the side walls or abutments. The semi-circular sections obviate part of this difficulty by omitting the side walls and resting the arch directly on the invert or foundation. In a rock trench the ability of the sides of the trench to resist pressure is so great that the side walls of the sewer can be greatly reduced in thickness, the thrust of the arch being carried directly into the rock. In this case a relatively flat arch can be used to advantage.

206. Imperviousness.—Where a sewer is to be constructed under a river bed or below the water table, it may be of particular importance for the walls of the sewer to be impervious. To this end, if the sewer is built of concrete, it is desirable to insert longitudinal reinforcing bars in the concrete, with a total area of

0.2 to 0.4 per cent of the sectional area of the concrete, in order to distribute the stress throughout the length of the sewer barrel and thereby prevent the formation of large cracks which would permit leakage. Unless the cracks are very small there may be some danger of corrosion due to the water passing through them and coming in contact with the reinforcement. This might in time weaken the structure.

While the possibility of leakage or infiltration does not ordinarily determine the shape of the waterway of a sewer, it is worthy of consideration when the selection is to be made. For example, if a sewer is to be built below the water table it may be well to adopt a section which is least likely to crack, whereas under other conditions the advantages of a different section might be sufficiently great to warrant its use even though small arch cracks were to be expected. The stability of the horseshoe section depends to a certain extent on the lateral pressure of the earth backfilling, and on that account, the semi-circular arch is apt to crack and may produce unsatisfactory conditions, not only because of leakage into the sewer, but especially on account of the rusting of the steel reinforcement. Expansion joints should be provided at suitable intervals, in order to provide for contraction without causing serious cracks.

207. Materials for Arches.—In constructing *brick* arches, two general types of bonding have been used. In the first, "rowlock bond," the arch is built of concentric rings of brick, all laid as stretchers. In the second type, part of the brick are laid as stretchers and part as headers, with radial joints in which the outer end of the joint is widened by increasing the thickness of the mortar or by inserting thin pieces of slate.

Plain concrete arches have been used to a considerable extent in recent years, and have an advantage over the stone or brick masonry arches in that the structure is somewhat more elastic and may withstand tensile stresses to a slight degree, although they should not be designed with this in view. In the design of such arches, as well as those of stone and brick, the line of pressure should fall within the middle third of the section, in order that no tensile stresses may be developed. If all the loads acting on the sewer were known exactly, it would be possible to design the section so that at no time would the line of pressure lie outside the middle third, but practically this is impossible, as our knowledge of the action of earth pressure is approximate only. On

that account, under special conditions the stresses in the arch section may not be entirely due to direct compression, but in addition bending stresses may be developed.

Arches of *reinforced concrete* are not subject to the limitations just mentioned, but can be made to withstand heavy bending moments by reinforcing the section with steel bars to carry tensile stresses. In arches in which the line of pressure lies within the middle third, the stresses in the arch are mainly due to compression and the concrete must of necessity carry the principal part of the load, so that the steel cannot be stressed to the allowable limit. On the other hand, the presence of the steel reinforcement furnishes a sort of insurance to the structure, to care for tensile stresses which may occur on account of unequal settlement of the foundations, temperature changes and many other conditions which cannot be foreseen. The steel also provides an additional factor of safety against careless and defective construction. On account of its presence, it is possible to increase slightly the allowable working stresses in the concrete over those which should be used for plain concrete masonry. Because of these considerations the authors believe that for large sewer arches reinforced concrete offers greater advantages than plain concrete, even though an analysis of the section shows that the line of resistance for the conditions considered will remain within the middle third of the masonry section.

208. Analysis of Stresses in Masonry Arches.—The complete analysis of the stresses in masonry arches is a highly complex problem which few engineers are ever called upon to solve. A considerable proportion of the large masonry sewers in the country were designed without anything more than a very elementary investigation of their arches. The increasing use of reinforced concrete sewers is responsible for a more general effort by designers to analyze the stresses in these structures, and now that a complete, accurate method of doing this has been developed, the design of large sewers can be made to depend upon both theory and experience instead of on the latter mainly. There are three general methods of analysis now in use, as follows:

The first method, called the "static analysis method," based on the so-called "hypothesis of least crown thrust," is applicable only to that portion of the sewer section above the springing line of the arch. Either the sewer must have very heavy side

walls or the thrust of the arch must be carried by the sides of a rock trench, in order that this method be strictly applicable.

The second method, based on the elastic theory of the arch and following the method described by Turneaure and Maurer in their "Principles of Reinforced Concrete Construction," is applicable to all sewer sections and can be used to cover all conditions. It has some mechanical disadvantages when applied to the analysis of the entire sewer structure, invert included.

The third method, also based on the elastic theory but using the so-called method for indeterminate structures, is of special advantage in the analysis of the entire sewer section as it permits a more suitable division of the axis in the side wall and invert. It does, however, require some additional labor over the second method when applied to an arch with fixed ends. For large sewers constructed in compressible soil and built of monolithic reinforced concrete, the third method is the most advantageous.

These methods are discussed in detail in the authors' "American Sewerage Practice," Vol. I, 2nd ed., Chap. XIV, to which the student is referred for further information.

209. Lining for Concrete Sewers.—From the observations made and tests conducted by the authors, it appears that in many cases where the estimated velocity of the sewage will be 8 ft. per second or greater when the sewer is full, and there is opportunity for hard grit, sand or gravel to enter the sewer, the invert may well be paved with hard-burned or preferably *vitrified paving brick* with square edges, laid with the edges projecting as little as possible and with full portland cement mortar joints. This invert paving should extend well up on the sides of the sewer, on straight sewers covering, in general, the arc of an angle of 90 deg. at the center of a circular sewer. The use of paving brick, as above suggested, is preferable to concrete, on account of the greater ease of making repairs and, further, on account of the probability that vitrified or even hard-burned brick will withstand the wear better than concrete of average quality. It is desirable when sewers are to be built of concrete to use hard aggregates, especially for inverts, and a first-class granolithic finish where the surface is subject to greatest wear is better than the ordinary concrete finish.

The use of vitrified-clay *liner plates* in concrete sewers has been introduced where excessive erosion is anticipated or the concrete is likely to deteriorate from the effect of acid or alkali in the sewage. They have been used not only in monolithic concrete sewers but also in reinforced concrete pipe of large

size. These tile liners are 9- by 18-in. plates with ¾ in. thickness and longitudinal dovetail ribs on the back to provide adequate bond with the concrete. They are made flat or with the face curved to a radius of from 16½ to 45 in. so as to be suited better to use in circular sewers. The curved plates are made in widths such that split plates are not required to construct the periphery of a sewer of given diameter. As such plates are difficult to cut to exact dimensions, it is advisable to procure sufficient plates of appropriate length to permit of carrying the lining around curves and of making closures without cutting.

There are two methods of laying the liner plates. They may be wired or otherwise attached to skeleton form work, thus making the sheathing against which the concrete is run. This is the usual method employed for precast pipe, vertical walls, roofs and arches. For floors and inverts, they are sometimes laid in cement mortar on the green concrete immediately after the removal of the forms. Satisfactory bond has been obtained by both methods. Irregularities in the tile make it impossible to obtain joints which are absolutely tight, so all joints should be pointed with cement mortar or other material.

To get the full advantage of tile liner plates, the joint should be as resistant to erosion and corrosion as the plates themselves. After three years of experimenting, the Los Angeles County Santitation Districts (A. K. Warren, Chief Engineer) decided to use for jointing a compound of sulphur, sand, and ground silica in the proportions of 10 parts sulphur, 7 parts clean sand passing a 50-mesh sieve, and 3 parts finely ground silica, by weight. This mixture, which becomes fluid at 250°F. and crystallizes rapidly at a somewhat lower temperature, is run into the joints between the plates which have been attached to the forms, using special apparatus for keeping the compound hot, and applying it to the joint from the outside.[1]

Problems

1. Determine the cross-sectional area required for a sewer to carry 50.0 c.f.s. when full if $S = 0.0005$ and $n = 0.013$, for each of the following sections: circular, egg-shaped, rectangular, semi-elliptical and semi-circular.

2. Determine the elevation of the water surface for each section in Problem 1 for discharges of 10.0, 25.0 and 40.0 c.f.s., assuming all inverts at an elevation of 100.00, and that Kutter's formula is applicable without change in the value of n.

[1] As described in *Eng. News-Rec.*, 1927; **99**, 346.

3. Compute the excavation in cubic yards per linear foot of trench required for each of the sections in Problem 1, if the hydraulic gradient is 10.0 ft. below the ground surface for the sewer when full. Allow a width of excavation of $1.4D + 24$ in.

4. If the above sewers are of reinforced concrete with an average thickness of 9 in., determine the quantity of concrete required in cu. yd. per linear foot of sewer, for each section.

CHAPTER IX

LOADS ON SEWERS

210. External Pressure on Pipes.—The external pressure or load carried by sewer pipes consists of that part of the weight of the earth above them which is transmitted to and carried by the pipes, together with so much of the weight of objects upon the surface (such as trucks or piles of building material) as is similarly transmitted.

Two classes of conditions are to be considered: (1) where a trench has been dug for the pipe and the pressure results largely or wholly from the weight of the back-filling material which is confined between the comparatively firm walls of the trench, and (2), where the pipe is laid on the surface of the ground, or in a trench of slight depth, and an embankment is then constructed over the pipe, in which case a much larger quantity of material contributes to the pressure on the pipe.

211. Investigations by Marston and Anderson.—Marston and Anderson use in their analytical treatment of pressures in trenches[1] practically the same method as that developed by Janssen for the pressures in grain bins.[2] This gives the weight on the pipe $W = cwB^2$, in which B is the width of the trench a little below the top of the pipe, and w is the weight per cubic foot of the backfill. The values of c are given in Table 53, and by the curves marked "Ditch Condition" (Fig. 108).

The approximate averages of a large number of measurements of weights and frictional properties of different classes of back-filling are given in Table 54. Within the range of ordinary ditch-filling materials, it takes a large difference in the values of the friction coefficients to make a material difference in the weight carried by the pipe. Marston and Anderson point out that the real difficulty in selecting the proper values from the

[1] The results of an elaborate investigation of the subject, lasting several years, were made public in 1913 in *Bull.* **31**, of the Engineering Experiment Station of the Iowa State College of Agriculture. This was written by Prof. Anson Marston, director of the station, and A. C. Anderson, and contains the first well-developed comprehensive theory of the subject which was also checked by numerous experiments.

[2] KETCHUM: "Retaining Walls, Bins and Grain Elevators."

table lies in deciding upon safe and reasonable allowances for the probable saturation of the materials under actual ditch conditions.

TABLE 53.—APPROXIMATE SAFE WORKING VALUES OF c IN THE MARSTON AND ANDERSON TRENCH-PRESSURE FORMULA $W = cwB^2$

Ratio of depth to width[1]	Values of c for			
	Damp top soil and dry and wet sand	Saturated top soil	Damp yellow clay	Saturated yellow clay
0.5	0.46	0.47	0.47	0.48
1.0	0.85	0.86	0.88	0.90
1.5	1.18	1.21	1.25	1.27
2.0	1.47	1.51	1.56	1.62
2.5	1.70	1.77	1.83	1.91
3.0	1.90	1.99	2.08	2.19
3.5	2.08	2.18	2.28	2.43
4.0	2.22	2.35	2.47	2.65
4.5	2.34	2.49	2.63	2.85
5.0	2.45	2.61	2.78	3.02
5.5	2.54	2.72	2.90	3.18
6.0	2.61	2.81	3.01	3.32
6.5	2.68	2.89	3.11	3.44
7.0	2.73	2.95	3.19	3.55
7.5	2.78	3.01	3.27	3.65
8.0	2.82	3.06	3.33	3.74
8.5	2.85	3.10	3.39	3.82
9.0	2.88	3.14	3.44	3.89
9.5	2.90	3.18	3.48	3.96
10.0	2.92	3.20	3.52	4.01
11.0	2.95	3.25	3.58	4.11
12.0	2.97	3.28	3.63	4.19
13.0	2.99	3.31	3.67	4.25
14.0	3.00	3.33	3.70	4.30
15.0	3.01	3.34	3.72	4.34

[1] The depth of trench is to the top of the pipe.

The approximate maximum loads on pipes in trenches of different widths and depths are given in Table 55. The investigations of Marston and Anderson have convinced them that a 12-in. pipe will have to carry the same load as an 18-in. pipe, if each is placed in the bottom of a 24-in. trench, other things being similar. When, for construction reasons, a wide trench

is necessary, they have shown that, in firm soil, the load on the pipe can be greatly diminished by stopping the wide trench a little above the top of the pipe and then excavating the narrowest trench in which it is practicable to lay the pipe, making special enlargements for the bells, if necessary.

TABLE 54.—PROPERTIES OF DITCH-FILLING MATERIALS
(Marston and Anderson)

Material	Weight of filling, lb. per cu. ft.	Ratio of lateral to vertical earth pressures	Coefficient of friction against sides of trench	Coefficient of internal friction
Partly compacted damp top soil	90	0.33	0.50	0.53
Saturated top soil	110	0.37	0.40	0.47
Partly compacted damp yellow clay	100	0.33	0.40	0.52
Saturated yellow clay	130	0.37	0.30	0.47
Dry sand	100	0.33	0.50	0.55
Wet sand	120	0.33	0.50	0.57

In commenting on Table 55, Marston and Anderson point out that the lateral pressure of the filling materials against the sides of the trench develops a frictional resistance which carries part of the vertical pressure near the sides of the trench, so that at the level of the top of the pipe, the vertical pressure of the filling materials is much greater in the middle of the trench than at the sides. Moreover, the side-filling material between the pipe and the sides is more compressible than the pipe and therefore can carry very little of the load. Hence, the pipe must have sufficient strength to carry the weight of all the backfill above the level of the top of the pipe, except that supported by friction upon the sides of the trench. Imperfections in the side filling and tamping probably increase the applicability of the principle.

Most analytical discussion of the pressures in trenches has been based upon the assumption of vertical sides. In many cases, the sides of the trench slope outward from its bottom, a condition which was investigated both analytically and experimentally by Marston and Anderson. An arching action appar-

ently takes place, they found, between the sides of the trench and points at the ends of the top quadrant of the pipe. Above the elevation of these 45-deg. points, the material along the sides settles less than that in the center of the trench. The investigations referred to led to the conclusion that in these wedge-shaped trenches the proper width to substitute for B in the formula

Table 55.—Approximate Maximum Loads, in Pounds per Linear Foot, on Pipe in Trenches, Imposed by Common Filling Materials (Marston and Anderson)

Depth of fill above pipe	Breadth of ditch at top of pipe									
	1 ft.	2 ft.	3 ft.	4 ft.	5 ft.	1 ft.	2 ft.	3 ft.	4 ft.	5 ft.
	Partly compacted damp top soil; 90 lb. per cubic foot					Saturated top soil; 110 lb. per cubic foot				
2 ft.	130	310	490	670	830	170	380	600	820	1,020
4 ft.	200	530	880	1,230	1,580	260	670	1,090	1,510	1,950
6 ft.	230	690	1,190	1,700	2,230	310	870	1,500	2,140	2,780
8 ft.	250	800	1,430	2,120	2,790	340	1,030	1,830	2,660	3,510
10 ft.	260	880	1,640	2,450	3,290	350	1,150	2,100	3,120	4,150
	Dry sand; 100 lb. per cubic foot					Saturated sand; 120 lb. per cubic foot[1]				
2 ft.	150	340	550	740	930	180	410	650	890	1,110
4 ft.	220	590	970	1,360	1,750	270	710	1,170	1,640	2,100
6 ft.	260	760	1,320	1,890	2,480	310	910	1,590	2,270	2,970
8 ft.	280	890	1,590	2,350	3,100	340	1,070	1,910	2,820	3,720
10 ft.	290	980	1,820	2,720	3,650	350	1,180	2,180	3,260	4,380
12 ft.	300	1,040	2,000	3,050	4,150	360	1,250	2,400	3,650	4,980
14 ft.	300	1,090	2,140	3,320	4,580	360	1,310	2,570	3,990	5,490
16 ft.	300	1,130	2,260	3,550	4,950	360	1,350	2,710	4,260	5,940
18 ft.	300	1,150	2,350	3,740	5,280	360	1,380	2,820	4,490	6,330
20 ft.	300	1,170	2,420	3,920	5,550	360	1,400	2,910	4,700	6,660
22 ft.	300	1,180	2,480	4,060	5,800	360	1,420	2,980	4,880	6,960
24 ft.	300	1,190	2,540	4,180	6,030	360	1,430	3,050	5,010	7,230
26 ft.	300	1,200	2,570	4,290	6,210	360	1,440	3,090	5,150	7,460
28 ft.	300	1,200	2,600	4,370	6,390	360	1,440	3,120	5,240	7,670
30 ft.	300	1,200	2,630	4,450	6,530	360	1,440	3,150	5,340	7,830
	Partly compacted damp yellow clay; 100 lb. per cubic foot					Saturated yellow clay; 130 lb. per cubic foot[1]				
2 ft.	160	350	550	750	930	210	470	730	1,000	1,240
4 ft.	250	620	1,010	1,400	1,800	340	840	1,330	1,870	2,370
6 ft.	300	830	1,400	1,990	2,580	430	1,140	1,900	2,630	3,410
8 ft.	330	990	1,720	2,500	3,250	490	1,380	2,360	3,360	4,400
10 ft.	350	1,110	2,000	2,920	3,880	520	1,570	2,760	3,980	5,270
12 ft.	360	1,200	2,220	3,320	4,450	540	1,730	3,100	4,560	6,050
14 ft.	370	1,280	2,410	3,650	4,950	560	1,850	3,410	5,050	6,760
16 ft.	370	1,330	2,570	3,950	5,400	570	1,940	3,660	5,510	7,440
18 ft.	380	1,380	2,710	4,210	5,810	570	2,020	3,880	5,930	8,060
20 ft.	380	1,410	2,830	4,450	6,180	580	2,090	4,070	6,280	8,610
22 ft.	380	1,430	2,920	4,640	6,500	580	2,140	4,240	6,610	9,130
24 ft.	380	1,450	3,000	4,820	6,800	580	2,180	4,380	6,910	9,590
26 ft.	380	1,470	3,060	4,980	7,080	580	2,210	4,500	7,160	10,010
28 ft.	380	1,480	3,120	5,100	7,310	580	2,240	4,610	7,380	10,430
30 ft.	380	1,490	3,170	5,230	7,530	580	2,260	4,700	7,590	10,780

[1] These two subtables contain the most important figures for practical use.

$W = cwB^2$ and to use as the width of the trench in Table 55 is the width at the height of the 45-deg. points on the pipe circumference.

212. Effect of Sheeting.—If sheeting is left in the trench, but the rangers removed, the friction between the backfill and the sides of the trench manifestly is decreased and the load on the pipe increased. The Marston and Anderson experiments indicate that this increase is from 8 to 15 per cent, and the experiments by Barbour[1] confirm this conclusion. If the rangers are left in place, the load coming on the pipes would probably be about the same as in unsheeted trenches, both according to theory and according to experiments by Barbour.

213. Earth Pressures.—In considering the loads on sewers due to the pressure exerted by the soil, it has been common practice to assume that the soil is a granular, cohesionless mass. This is only true for clean, dry sand while for clay, silts and similar soils, cohesion is an important factor. When soil is excavated, as for a sewer trench, and later replaced in backfill, its cohesion is disturbed and this effect cannot be relied upon for decreasing the load on the sewer.

Where such cohesion does not exist, the maximum surface slope that the soil will assume will depend upon the coefficient of internal friction. The vertical angle between the surface slope and the horizontal is called the angle of repose, and its tangent equals the coefficient of friction. This coefficient varies with the character of the soil and its moisture content. Where the coefficient of friction is zero, as with water, the resultant pressure against any plane is normal to that plane and can have but one value consistent with equilibrium. Where internal friction exists, however, as in soils, the resultant may make an angle with the normal to the plane, which may be above or below that normal, depending on the direction in which the friction is assumed to act. Its magnitude may vary from zero to the angle of repose. The latter case gives the lower and upper limits to the possible values of the resultant which satisfy the conditions of equilibrium. The lower value is called the *active earth pressure*, while the higher is the *passive earth pressure*. The *active pressure* is that which the soil exerts against a retaining wall or sewer; the *passive pressure* is the resistance of the soil to displacement by an external force.

[1] *Jour.*, Assoc. Eng. Soc. 1897; **19**, 193.

Earth pressures are usually calculated by Rankine's formula for cohesionless soils. Where cohesion exists, a coefficient of from 0.25 to 1.0 may be applied, depending upon the character of the soil. The formula is:

$$P = \frac{wh^2}{2} \cos \phi \times \frac{\cos \theta \mp \sqrt{\cos^2 \theta - \cos^2 \phi}}{\cos \theta \pm \sqrt{\cos^2 \theta - \cos^2 \phi}}$$

Notation:

w = weight of a unit volume of earth
P = resultant pressure of the mass of earth against a vertical surface
H = horizontal component of P
ϕ = angle of repose of a particular earth
θ = angle which the surface of the ground makes with the horizontal
h = depth of any point on the vertical surface resisting the earth pressure
p = intensity of the horizontal pressure at any depth, h, below the surface.

The active pressure is obtained by using the upper signs, while the passive pressure is obtained with the lower.

If the plane against which the lateral pressure is exerted is vertical, the resultant earth pressure, P, will be parallel to the surface of the ground.

Where the angle θ is equal to the angle of repose ϕ, the formula becomes

$$P = \frac{wh^2}{2} \cos \phi$$

If there is no surcharge and the surface is horizontal, θ will equal zero, and the formula becomes,

$$P = H = \frac{wh^2}{2} \times \frac{1 - \sin \phi}{1 + \sin \phi}*$$
$$= \frac{wh^2}{2} \tan^2 (45 - \frac{\phi}{2})$$

If it is assumed that the direction of the resultant pressure P is parallel to the surface of the ground

$$H = P \cos \theta$$

* Values of the last factor are given in Table 56.

If the angle of repose is zero, as in the case of a liquid, the formula becomes

$$P = H = \frac{wh^2}{2}$$

In all cases P varies as the square of the depth, as in the case of a liquid, and it has therefore been assumed by most engineers that the resultant earth pressure, P, is applied at a point two-thirds the depth. Recent experiments indicate that this may vary from 0.60 to 0.67 of the depth. The intensity of pressure at any point may be found by the formula

$$p = \frac{2P}{h}$$

It is frequently assumed in calculations of this kind that the angle of repose is 30 deg. The formula then becomes,

$$P = H = \frac{wh^2}{6}$$

and

$$p = \frac{wh}{3}$$

These last formulas are simple and for practical purposes are all that need be used to obtain a general idea of the earth pressures involved. Apparatus has been developed with which the friction factor can be measured in the laboratory for a particular soil. This factor is then used in the formula instead of the estimated value above mentioned.

The angle of repose of many different earths has been determined, but the reported values are of only general applicability because of the absence of any method of describing the characteristics of earth so definitely that exactly what material is meant by a description is clearly known. Rough average values are given in Table 56, taken from Baker's "Treatise on Masonry Construction," supplemented with a column of factors for use in the Rankine formula for pressures when the ground surface is horizontal.

Rankine's formula is frequently used in computing pressures of earth loads upon sewers, in which case the surface of the earth is usually assumed to be horizontal. The total earth pressure acting on a section of a sewer arch may be considered as composed of a vertical component equal to the weight of the column of

earth above the section and a horizontal component which at any point cannot be greater than $(1 + \sin \phi)/(1 - \sin \phi)$ times the vertical pressure at the same point, nor less than $(1 - \sin \phi)/(1 + \sin \phi)$ times the vertical pressure, ϕ being the angle of repose. The former expression represents the passive resistance of the earth, while the latter represents the active pressure, which probably is realized. If the angle ϕ is taken as 30 deg., which is a convenient figure to use and approximately represents average conditions, the above statement means that the horizontal pressure cannot be greater than three times the vertical pressure nor less than one-third of it. While it is recognized that a more logical course would be to use exact values for the angle of repose, or, better, the angle of internal friction, this is hardly justified for ordinary conditions because of the great uncertainty regarding the action of earth pressures and the variation in the character and condition of trench materials.

TABLE 56.—ANGLE OF REPOSE AND WEIGHT OF SOILS

Kind of earth	Angle of repose		$\frac{1 - \sin \phi}{1 + \sin \phi} = \tan^2\left(45 - \frac{\phi}{2}\right)$	Weight, lb. per cu. ft.
	ϕ	Slope[1]		
Alluvium	18°	3 to 1	0.53	90
Clay, dry	26°	2 to 1	0.39	110
Clay, damp	45°	1 to 1	0.17	120
Clay, wet	15°	3.2 to 1	0.59	130
Gravel, coarse	30°	1.7 to 1	0.33	110
Gravel, graded sizes	40°	1.2 to 1	0.22	120
Loam, dry	40°	1.2 to 1	0.22	80
Loam, moist	45°	1 to 1	0.17	90
Loam, saturated	30°	1.7 to 1	0.33	110
Sand, dry	35°	1.4 to 1	0.27	100
Sand, moist	40°	1.2 to 1	0.22	110
Sand, saturated	30°	1.7 to 1	0.33	120

[1] Ratio of horizontal to vertical dimension.

214. Terzaghi's Studies.—It should be mentioned that the methods of Rankine and Coulomb, for determining earth pressures, have been called into question by Dr. Charles Terzaghi in numerous articles, from about 1920 to date,[1] and he has

[1] *Eng. News-Rec.*, 1920; **85**, 632; 1925; **95**, 742, 796, 832, 874, 912, 987, 1026.

suggested new theories for the behavior of earths based upon their elastic properties. "Slip" is held to be an incidental rather than an essential event, since an appreciable deformation or movement of the wall or arch is required before "slip" can occur. As the structure yields, the intensity of the pressure is decreased. For example, with cohesionless sand where, according to accepted theories,

$$P = \tfrac{1}{2}cwH^2$$

c is 0.42 before deformation of the structure, while, after such yielding begins, c ranges from 0.15 to 0.05 or less.

Terzaghi's method of determining earth pressures is based on tests for compressibility, permeability, and various other characteristics of the earth in question. The method of utilizing these data is not yet sufficiently formulated for general application but requires considerable understanding of the nature and behavior of various kinds of earths. It is to be hoped that the increasing amount of research that is being carried out in various quarters will present sufficient data on earths of all kinds to permit a more logical and accurate method of determining earth pressure for which provision should be made.

215. Mohr's Method of Determining Pressures.—A graphical method of determining earth pressures, devised by Professor Mohr in 1871 and founded on Rankine's theory, was described by Prof. G. F. Swain[1] as follows:

Let a horizontal line AH (Fig. 106) represent the surface of the earth. Draw HI perpendicular to AH, and of some convenient length, as 5 in., equivalent to 10 ft. on a scale of ½ in. to 1 ft. Lay off

$$HK = HI \tan^2 (45° - \tfrac{1}{2}\phi)$$

where ϕ = angle of repose. This will be recognized as equivalent to Rankine's formula for the intensity of earth pressure, with w, the unit weight of earth, omitted.

$$P = wH \tan^2 (45° - \tfrac{1}{2}\phi)$$

where P = the total earth pressure per unit length of wall or sewer, by Rankine's theory, and H = the depth of earth.

Having located point K, with KI as a diameter, describe a circle. Through I draw a line IV_1 parallel to the face of the wall or section of arch upon which the pressure of the earth acts. Draw V_1K through the points V_1 and K on the circumference of the circle, and prolong it to meet the surface line AH. At this point of intersection A, draw AI, which

[1] *Jour.* Franklin Inst., 1882; **114**, 241.

gives the direction of the active pressure on plane 1. The distance HV_1 measures the magnitude of this pressure for the depth represented by HI. $(HV_1,HI)w$ is the intensity of active pressure per unit depth of earth on plane 1. The magnitude of HV_1 can be obtained by scaling the line HV_1. In a similar manner, the direction and amount of the active pressure on any other plane, as plane 2, can be found.

The amount of the maximum passive earth pressure is measured by HI for a depth of HV_1 (HI/HV_1 = intensity) for plane 1, or by HI for a depth of HV_2 for plane 2. The direction of the maximum passive pressure is found by drawing through V_1, V_2, etc., a diameter of the circle and then

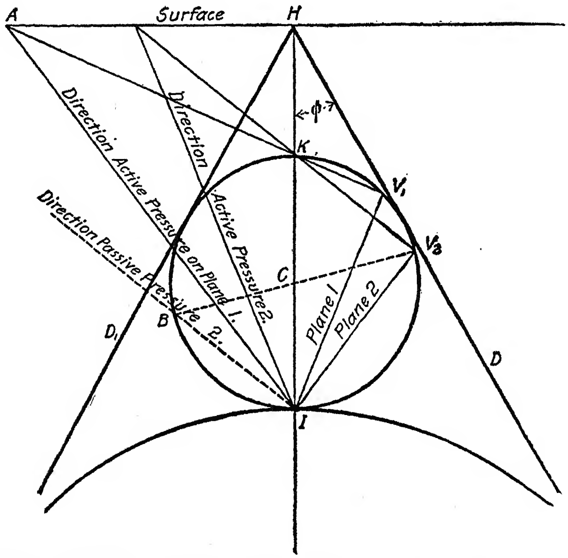


FIG. 106.—Mohr's graphical method of determining earth pressures.

connecting the point of intersection B with I. Line BI is the direction of the maximum passive pressure for plane 2. It is perpendicular to the face upon which pressure is exerted.

There is an exact mathematical proof of the foregoing, but the following general proof will probably be sufficient:

If, in the figure, we let the line HI represent a vertical plane, we have chosen HK of such a distance that for the depth HI, HK represents the intensity of the active earth pressure.

It can be proved that as the plane of the wall slants away from the vertical, a circle of diameter KI will contain all the points V for every position of the plane, the intensity being HV/HI until a horizontal surface is reached which has a pressure of $HV/HI = HI/HI = 1$, or the total dead weight of the earth above the plane. The angle IHV is the angle S, or the angle which the direction in which pressure acts makes with the normal to the plane.

From Rankine's theory, we know that the angle S can never exceed the angle of friction ϕ, or the angle of repose of the earth. Hence, if we draw from H two lines making angles of ϕ on either side of HI, we know the circle must lie within those lines, and when the earth is just on the point of slipping, $S = \phi$ and the circle is tangent to the two lines HD and HD'. There are two circles which satisfy the conditions representing the two limiting states of equilibrium when the earth is just ready to slip. The larger circle, only part of which is shown in Fig. 106, represents the case where the maximum principal pressure HI is increased until the limiting condition is reached. This is the passive earth pressure. The smaller circle represents the case where the minimum principal pressure HK is decreased until the limiting condition is reached. This is the active earth pressure. In the case of $\phi = 30$ deg., for which the figure is drawn, the passive earth pressure is nine times the active. It is not necessary, however, to use the large circle, since for the active pressure

$$P_a = wH \frac{1 - \sin \phi}{1 + \sin \phi}$$

and for the passive pressure

$$P_p = wH \frac{1 + \sin \phi}{1 - \sin \phi}$$

the term $(1 - \sin \phi)/(1 + \sin \phi)$ being merely inverted. The inversion has been accomplished as follows:

The active pressure per unit depth $= w(HV/HI)$

The passive pressure per unit depth $= w(HI/HV)$

The angle $IHV =$ the angle S, the angle between the normal to the plane and the direction in which pressure acts. Therefore, having this angle, we can erect a normal to the plane and lay off the angle S, thereby obtaining the direction of the stress. For example:

$$\text{angle } IAV_1 = \text{angle } IHV_1$$

Recent experiments (1926) at the Iowa Experiment Station[1] on the supporting strength of pipes in embankments "confirm the general correctness and reliability of Rankine's formula for calculating the active horizontal pressure in masses of granular materials."

SURFACE LOADS TRANSMITTED TO SEWERS

216. Live Loads.—Sewers constructed in shallow cut are often subjected to the effect of loads on the surface, transmitted through the earth filling. If the sewer line is crossed by steam-railroad tracks, there will be heavy loads from locomotives or loaded freight cars; if crossed by an electric railroad, there will be loads from passenger or express cars, construction cars, or snow plows, which, in the case of high-class interurban lines, amount to

[1] *Bull.* 76, Engineering Experiment Station, Iowa State College, Ames, Iowa.

approximately the same as the loads on second-class steam railroads. In highways, sewers are subject to the loads of steam road rollers, traction engines, and heavy trucks.

TABLE 57.—STANDARD LOCOMOTIVE LOADINGS

STANDARD LOCOMOTIVE LOADINGS.

		Axle 1	Axle 2	Axle 3	Axle 4	Axle 5	Axle 6	Axle 7	Axle 8	Axle 9	
Cooper[1]	Axle Spacing, Ft.	8'	5'	5'	5'	9'	5'	6'	5'		
Cooper[1] Class E-30	Axle Load	15,000	30,000	30,000	30,000	30,000	19,500	19,500	19,500	19,500	Uniform Load 3,000 Lb per Lin. Ft.
Cooper[1] Class E-40	Axle Load	20,000	40,000	40,000	40,000	40,000	26,000	26,000	26,000	26,000	Uniform Load 4,000 Lb. per Lin. Ft
Northern Pacific Ry[2]	Axle Load	25,000	52,000	52,000	52,000	52,000	36,000	36,000	36,000	36,000	Uniform Load 5000 Lb. per Lin. Ft.
Atchison, Topeka & Santa Fe Ry[2] Heavy Grade	Axle Spacing in Feet	7.5'	4.5'	4.5'	4.5'	10.5'	5'	5.5'	5'		
Atchison, Topeka & Santa Fe Ry[2] Heavy Grade	Axle Load	27,000	66,000	66,000	66,000	66,000	30,000	30,000	33,000	33,000	Uniform Load 4,800 Lb. per Lin. Ft.

[1] From Cooper's General Specifications for Steel Railroad Bridges.
[2] From *Trans. Am. Soc. C. E.*, 1905; **54A**, 82.

For convenience in estimating live loads, four tables are given: Table 57, typical standard locomotive axle loads and spacings; Table 58, typical axle loads of cars for heavy freight, such as coal or iron ore; Table 59, typical axle loads of the heavy type of electric cars for suburban service; and Table 60 the wheel loads and general dimensions of steam road rollers, traction engines and heavy automobile trucks.

TABLE 58.—TYPICAL HEAVY FREIGHT CARS[1]

TYPICAL HEAVY FREIGHT CARS

		Axle 1	Axle 2	Axle 3	Axle 4
Steel Coal Cars	Axle Spacing in Feet-Inches.	5'6"	19'9"	5'6"	
Steel Coal Cars	Axle Load	35,000	35,000	35,000	35,000
Iron Ore Cars	Axle Spacing in Feet-Inches.	5'6"	17'9"	5'6"	
Iron Ore Cars	Axle Load	60,000	60,000	60,000	60,000

[1] *Trans. Am. Soc. C. E.*, 1905; **54A**, 85.

Locomotives have increased in weight to such an extent that most main-line railroad bridges are now designed for Cooper's Class E-50 loading, or heavier, which is five-fourths times the Class E-40 loading, shown in Table 57, on the same axle spacing.

TABLE 59.—TYPICAL HEAVY ELECTRIC CARS

Car		
Long Island R.R. 1907 — 53 Tons[1]	Axle Spacing Feet	5.45′ – 5.5′ – 27.9′ – 6.7′ – 5.45′; 51.0′
	Axle Load Pounds	30,800 – 30,800 – 22,200 – 22,200
Boston Elevated Ry. Elevated Car No. 3 40.5 Tons[2]	Axle Spacing Feet	6.0′ – 26.25′ – 6.08′; 46.60′
	Axle Load Pounds	16,920 – 16,920 – 23,620 – 23,620
Boston Elevated Ry. Cambridge Subway Car 62.9 Tons[2]	Axle Spacing Feet	6.0′ – 44.5′ – 7.0′; 69.21′
	Axle Load Pounds	28,000 – 28,000 – 34,900 – 34,900
Interborough Rapid Transit Co. Manhattan Ry. Div. Car 38.0 Tons[2]	Axle Spacing Feet	5.0′ – 27.67′ – 6.0′; 47.33′
	Axle Load Pounds	15,850 – 15,850 – 22,150 – 22,150
Interborough Rapid Transit Co. Subway Division Car 55.98 Tons[2]	Axle Spacing Feet	5.5′ – 29.92′ – 6.67′; 51.33′
	Axle Load Pounds	24,240 – 24,240 – 31,740 – 31,740
Bay State St. Ry. Co. Express Car 49.5 Tons[2]	Axle Spacing Feet	6.33′ – 15.67′ – 6.33′; 42.0′
	Axle Load Pounds	24,750 – 24,750 – 24,750 – 24,750
Typical Passenger Car Trans. Am. Soc. C.E. 1924; 87, 1277	Axle Spacing Feet	7.0′ – 27.0′ – 7.0′
	Axle Load Pounds	30,000 – 30,000 – 30,000 – 30,000
Typical Freight Car Trans. Am. Soc. C.E. 1924, 87, 1277	Axle Spacing Feet	6.0′ – 19.0′ – 6.0′
	Axle Load Pounds	50,000 – 50,000 – 50,000 – 50,000

1. *From Jour. Assn. Eng. Soc. 1907; 43, 241* 2. *From Proc. Eng. Soc. West. Penn. 1915; 31, 738 & 742*

Additional data regarding loads from electric cars, auto trucks, and occasional heavy loads to which highways are subjected can be found in a paper by Charles M. Spofford.[1]

In the *Second Progress Rept.* of the Special Committee to Report on Stresses in Railroad Track,[2] the results are given of tests of the distribution of pressure in the ballast under loaded railroad ties. For the standard track construction the vertical pressure at a depth of 30 in. below the underside of the ties was practically uniform for equal loads on the ties and in intensity

[1] Highway Bridge Floors. *Proc.*, Eng. Soc. Western Penn., 1915; **31,** 727.

[2] *Trans.*, A. S. C. E., 1919–1920; **83,** 1573.

amounted to 33 per cent of the average intensity of pressure applied to the ballast by the tie.

TABLE 60.—WEIGHTS OF ROAD ROLLERS, TRACTORS, AND TRUCKS

Rating	Total weight equipped in pounds	Load per wheel in pounds	Diameter of wheels in inches		Face width of wheels in inches		Distance c. to c. of axles		Width of track, in.
			Front	Rear	Front	Rear	Ft.	In.	
Weights of steam road rollers (*Data Furnished by the Buffalo Steam Roller Co.*)									
10 tons	26,000	8,670	44	69	47½	18	9	10	
12 tons	31,000	10,340	46	69	51	20	10	8	
15 tons	39,000	13,000	48	72	52½	22	11	1	94
Weight of traction engine (*Data Furnished by the Good Roads Machinery Co.*)									
16 h.p.	19,580	6,530[1]	40	66	12	19	10	6	82
Weights of typical automobile trucks									
5 tons	20,000[2]	6,900[1]	36	42	6	13	12	6	86
20 tons[3]	40,000	16,000[1]	..	..	....	20	14	0	92

[1] Rear wheels.
[2] Allows for 25 per cent overload.
[3] *Trans.*, A. S. C. E., 1924; **87**, 1277.

The loads from the wheels of steam road rollers, traction engines, trucks, etc., are applied directly to the surface of the fill but over a very small area. Although the intensity of the load at the surface is great, it becomes distributed fairly well over the entire width of the trench at a depth of 5 ft. or more and in a similar manner longitudinally.

217. Dead Loads.—In manufacturing districts, sewers are often subjected to heavy surface loads from piles of lumber, brick, pig iron, coal, etc. Wherever such is likely to be the case, ample allowance should be made. It is not uncommon to find surface loads as high as the following: lumber, 850 lb. per square foot; brick, 900 lb.; coal, 1,200 lb.; and pig iron, 2,300 lb.

There are cases, doubtless, where heavy masonry foundations have been built over sewers without regard for their stability. Wherever it is necessary to do such work, either the sewer arch should be strengthened to carry the excess load, or, preferably,

TABLE 61.—ESTIMATED INTENSITIES OF SURFACE LOADS

Loading	Assumed pressure on under side of ties, assuming one axle load on the least number of ties between axles, pounds per square foot	Estimated equivalent intensity of load on a plane 30 in. below underside of ties, pounds per square foot
Locomotive, Cooper's Class E-30	1,875	620[1]
Locomotive, Cooper's Class E-40	2,500	825
Locomotive, Cooper's Class E-50	3,125	1,030
Locomotive, Northern Pacific R. R.	3,250	1,070
Locomotive, A. T. & S. F. R. R.	6,180	2,040
Steel coal car	2,190	720
Steel ore car	3,750	1,240
Electric car, Long Island R. R.	1,925	635
Electric car, Boston & Worcester St. Ry.	1,560	515
Electric car, Boston Elev. Ry. Elevated Car No. 3	1,480	490
Electric car, Boston Elev. Ry., Cambridge Subway Car	2,120	700
Electric car, Interborough Rap. Tr. Co. Man. Ry. Div. Car	1,385	460
Electric car, Interborough Rap. Tr. Co. Subway Div. Car	1,985	655
Electric car, Bay State St. Ry. Co. Express Car	1,550	510
Electric car, Typical Passenger Car	1,875	620
Electric car, Typical Freight Car	3,125	1,030
	Estimated equivalent load on surface, pounds per linear foot of trench[2]	
Steam road roller 10 tons	8,670	
Steam road roller 12 tons	10,340	
Steam road roller 15 tons	13,000	
Traction engine 16 hp	6,530	
Automobile truck 5 tons (rating)	6,900	
Automobile truck 20 tons (total weight)	16,000	

[1] Assumed intensity of pressure 33 per cent of that on underside of ties.

[2] Assuming weight of one wheel per linear foot of trench. If trench is wide enough to receive both rear wheels, load assumed should be that upon the two rear wheels.

the foundation in question should be built so as to relieve the sewer arch of all of the load of the building or structure.

218. Proportion of Loads Transmitted to Sewers; Investigations of Marston and Anderson.—In order to determine the effect of such excess loads, Marston and Anderson[1] carried out an analytical and experimental investigation. They found that for a long load extending along the trench, the decimal part of it which is transmitted to the pipe in trenches of different dimensions is approximately that given in Table 62.

TABLE 62.—PROPORTION OF "LONG" SUPERFICIAL LOADS ON BACK-FILLING WHICH REACHES THE PIPE IN TRENCHES WITH DIFFERENT RATIOS OF DEPTH TO WIDTH AT TOP OF PIPE

(Marston and Anderson)

Ratio of depth to width	Sand and damp top soil	Saturated top soil	Damp yellow clay	Saturated yellow clay
0.0	1.00	1.00	1.00	1.00
0.5	0.85	0.86	0.88	0.89
1.0	0.72	0.75	0.77	0.80
1.5	0.61	0.64	0.67	0.72
2.0	0.52	0.55	0.59	0.64
2.5	0.44	0.48	0.52	0.57
3.0	0.37	0.41	0.45	0.51
4.0	0.27	0.31	0.35	0.41
5.0	0.19	0.23	0.27	0.33
6.0	0.14	0.17	0.20	0.26
8.0	0.07	0.09	0.12	0.17
10.0	0.04	0.05	0.07	0.11

NOTE.—Curves based on this table are given in Fig. 107.

Where the load is imposed by some short object, such as a road roller, the results of the investigation are not so positive, for it was found impracticable to test the theory upon which the analysis of such conditions was based. This theory was about the same as that found to be correct in other work when tested experimentally, so the results in this case are of considerable value even if purely theoretical. Apparently, the proportion of the load reaching the pipe depends on the ratio of the length of load along the trench, the width of the trench and the ratio of the lateral and longitudinal pressures in the backfilling.

[1] *Bull.* **31**, Engineering Experiment Station, Iowa State College of Agriculture and Mechanic Arts, 1913.

The curves in Fig. 107 will be found of value in estimating the proportion of the weight of surface loads, both long and short, that might be transmitted through the backfilling to the sewer. All such loads, after having been reduced in the proportion shown by the curves or as aided by judgment, should be added to the load due to the backfill obtained by the method previously described. In this way the backfilling and surface loading will be reduced to the load in pounds per linear foot on the pipe.

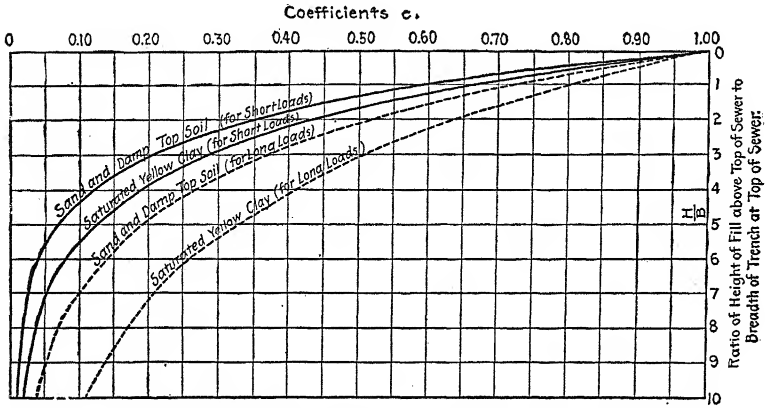


FIG. 107.—Coefficients of surface loads transmitted through earth fill in trench to sewers.

The allowance for increase in the load on the pipe due to impact in the case of the faster moving short loads will depend largely on the judgment of the engineer. It is believed, however, that for ordinary highway conditions, an increase of 50 to 100 per cent over the calculated load will be sufficient.

For example, the depth of earth over the crown of a certain sewer is 20 ft., and the width of trench at the top of the sewer is 10 ft. The backfilling material is sand weighing 120 lb. per cubic foot. One section of this sewer is to be built under a coal yard, and accordingly there should be added a surface load due to piles of coal of 1,200 lb. per square foot. The total "long load" per linear foot of trench, W_S, would be $1{,}200 \times 10 = 12{,}000$ lb. The ratio of height of fill to width of trench, $H/B = 2$. On Fig. 107, follow along the horizontal line $H/B = 2$ until it intersects the curve for sand and damp top soil for long loads, which point is on the vertical line (interpolated) for coefficient $c = 0.52$. Substituting in the formula $W_P = cW_S$ the values of $c = 0.52$ and $W_S = 12{,}000$, we have $cW_S = 0.52 \times 12{,}000 =$ 6,240 lb. per linear foot of sewer.

In order to determine the total load on the pipe it is necessary to obtain the load due to the backfill. The value of H/B is 2 as previously, and from Table 53, c in the formula $W = cwB^2$ would be 1.47. Then $W = 1.47 \times 120 \times 10^2 = 17{,}620$ lb. per linear foot due to the backfill. The total load on the pipe will be $17{,}620 + 6{,}240 = 23{,}860$ lb. per linear foot.

219. Other Experiments.—A. T. Goldbeck[1] and Prof. M. L. Engèr[2] have also made experiments which are of interest in this connection, although these experiments are not extensive enough to permit of direct application to sewer design.

220. Possible Load on Pipe Resulting from Tamping Backfill.—An example of the possible use of Fig. 107 is given by Marston and Anderson in a discussion of the probable correctness of the general impression that pipes with a small depth of cover are susceptible to greater damage than those in deep trenches and that more damage is done during tamping than is frequently considered probable by those who write specifications for pipe. The maximum pressure V_s on the backfill, resulting from the shock of a blow of a rammer, is $2TF/f$, where T is the weight of the rammer in pounds.

The data for an example of the use of the formula may be taken from a discussion by J. N. Hazlehurst. Here the original "very thorough" tamping was done with a 40-lb. rammer on a 6-in. clay cover, resulting in some cracking of the pipe, while, later, the use of a 30-lb. rammer on a 12-in. fill had no such result. If it be assumed that very thorough tamping on a 6-in. cover is such as would produce a final compression f of 0.01 ft. under one blow, and the height of fall F was 0.5 ft., then, with a 40-lb. rammer, $V_s = 4{,}000$ lb. If the rammer had a face width b, of 0.67 ft., then the ratio of depth of cover to the width over which the load was applied, $H/b = 0.5/0.67$, was 0.75. The percentage of V_s reaching the pipe would be, from Fig. 107, about 73. Hence, about 2,920 lb. would be directly transmitted to an 8- by 8-in. area of pipe. With the lighter rammer, f would probably be a little larger, say 0.015 ft., because the cover was 1 ft. instead of 0.5 ft. The same method of computation as in the first case shows that the pressure on the 8- by 8-in. area would be about 1,080 lb. The correctness of the opinion occasionally expressed, that the use of a rather thick cover and light rammer in the lower part of the trench is desirable for the safety of the pipe, is confirmed by this analytical method of investigation.

[1] *Proc.*, A. S. T. M., 1917; **17**, Part II, 641.

[2] *Eng. Rec.* 1916; **73**, 106.

PRESSURE IN EMBANKMENTS

221. Marston's Culvert-pipe Investigations.—Beginning in 1915, Prof. Anson Marston has carried out an extensive investigation of the loads transmitted to culvert pipe, meaning thereby pipe which were laid on or slightly below the natural surface of the ground and then covered by an embankment. This is the usual condition experienced in building culverts. It is not uncommon in sewer work, particularly for outfall or intercepting sewers, or when the sewers are constructed at the same time as highway embankments.

The experiments were made in a similar way to those on pressure in trenches. It was found[1] that the same expression as for pressure in trenches was applicable, namely,

$$W = cwB^2$$

in which B is the greatest outside width of the pipe or culvert, while c is considerably greater than for pipes in trenches.

The tests showed that if the material of the embankment was consolidated by wetting and rolling, to an elevation above that of the top of the culvert amounting to at least one-third of the total height of the embankment above that point, and a trench were then excavated in the compacted earth and the culvert built, the resulting loads upon the pipe after the embankment was completed were the same as though the pipe had been laid entirely in a trench; in other words, "ditch conditions" prevailed, and the coefficients given in Table 53 were applicable. If the pipe were laid first, either on or somewhat below the natural surface, and the embankment built over it—called "projection condition," since a part of the pipe projects above the original surface—it was found that coefficients depending, in part, upon the extent of such projection were applicable, as already noted.

If the lower part of the embankment was constructed and consolidated before laying the pipe, but to a height above the top of the culvert less than one-third of the total height of the embankment above that level, then "imperfect ditch conditions" existed, and the values of the coefficient were uncertain but somewhere between those for "projection conditions" and "ditch conditions."

[1] *Second Progress Rept.* on Culvert Pipe Investigations, Iowa Eng. Experiment Station, 1922.

Figure 108[1] shows graphically the values of the coefficient c for ditch conditions, for projection conditions with various

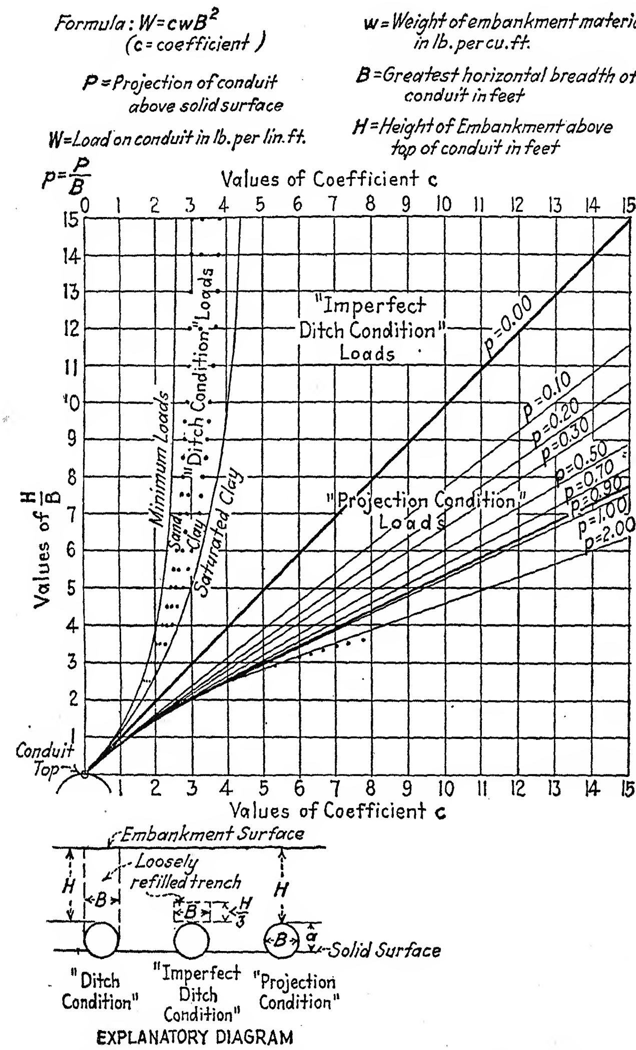


FIG. 108.—Computation diagram for loads on conduits. (*Marston.*)

amounts of projection, and for the intermediate zone of "imperfect ditch conditions."

[1] Reproduced from MARSTON'S *Rept.*

The experiments were carried out with embankments of various heights, up to 20 ft. The applicability of the coefficients to conditions in which the height of embankment materially exceeds 20 ft. is, therefore, open to question.

The horizontal pressure exerted by the material of the embankment is best determined by Rankine's formula given on p. 302.

222. Weight of Backfilling.—For much designing work, it is sufficient to assume that the backfill will weigh 120 lb. per cubic foot and that the horizontal pressure at any depth due to this fill will be one-third of the vertical pressure. Where more precise figures are required, the material which will be used should actually be weighed, in a moist as well as a dry condition, or the information given in Table 54, p. 299 should be employed.

Problems

1. With 20 ft. fill over the crown of a 48-in. circular sewer in clay soil, find the load per foot of sewer due to the backfill if the trench width is 5 ft.

2. After excavating the trench for the sewer in Problem 1, it is decided to use an egg-shaped section of equivalent area. Determine the load per foot of sewer due to the backfill.

3. Draw the sewer section in Problem 1 to suitable scale and indicate graphically the unit pressures exerted by the surrounding soil, both as to magnitude and direction.

4. (*a*) A rectangular sewer 6 ft. high outside dimensions has a 10 ft. cover of moist loam. Compute the resultant earth pressure acting against its sidewalls and indicate its direction and point of application. (*b*) Recompute for a wet clay backfill.

5. Check the determination of Problem 4 (*a*) using Mohr's graphical method.

6. (*a*) A 36-in. cast iron sewer passes under a railroad embankment so that the crown is 5.0 ft. below the bottom of the ties. If the soil is damp sand and top soil, determine the maximum load reaching the sewer assuming that Cooper's Class E-40 represents the locomotive loading to be anticipated.

(*b*) Ditto for Cambridge subway car as the live load.

(*c*) Ditto for a 5-ton automobile truck.

7. A 24-in. vitrified pipe culvert is constructed through a fill by cutting out a 6-in. sector in the natural soil to fit the under side of the pipe. The backfill is then placed and tamped, to a height of 10 ft. above the crown of the culvert, and weighs 120 lb. per cubic foot. Determine load to be carried per foot of culvert.

CHAPTER X

EXCAVATION, SHEETING AND BRACING OF TRENCHES

EXCAVATION

Excavation is that part of the work of sewer construction which is usually most important in determining costs. Good management and judgment are required in handling this work to advantage. While the materials used in sewer construction generally represent a considerable part of the total cost of the project, the contractor's profit on these is not great. The chief source of profit is in so handling the work that the labor and machinery charges are kept at a minimum. The seasonal element is of considerable importance, since work can be pushed more rapidly during the summer and costs kept at a minimum, thus avoiding considerable expense which may result if work is prolonged into the winter. Charges for plant operators and machinery run up the unit costs rapidly when progress is slow. An allowance for delays due to unfavorable weather, particularly rain and cold, is necessary. This factor is beyond the control of the sewer builder and is an important one in determining profit or loss on contracts.

223. Classification.—There is little agreement among sewerage engineers as to the classification of excavated materials for purposes of payment. The prevailing opinion among engineers, in the eastern states, whose work is with materials in which the rock is usually hard and fairly sharply distinguishable, is that a classification into rock and earth should be made. Among engineers in the West it appears customary to use no classification. The authors use the following classification for rock:

Rock, wherever used as the name of an excavated material, shall mean boulders exceeding ½ cu. yd. in volume, or solid ledge rock which, in the opinion of the engineer, requires for its removal drilling and blasting, or wedging, or sledging or barring. No soft or disintegrated rock which can be removed with a pick; no loose, shaken or previously blasted rock or broken stone in rock fillings or elsewhere, nor rocks exterior to the maximum limits of measurement allowed which may fall into the trench, will be measured or allowed as rock.

224. Opening the Trench.—Where paving materials encountered in excavation are to be used again, they should be placed in neat piles outside the probable limits of excavation. Other excavated material is placed in a windrow along one side of the trench.

Frost enters compacted ground to a greater depth than where it is in its natural condition. Where frost penetrates to a depth of 3 ft. in a field, it may penetrate 5 or 6 ft. in a neighboring travelled road.[1] Opening a trench in frozen ground is slow work and it is common to first thaw the surface by building fires at the desired points. Thawing may be done by steaming, first covering the surface with long wooden boxes about 10 in. high and as wide as the trench, with the open side of the box down and holes bored in the top about every 12 in. Each hole will be supplied with a wooden plug to prevent the escape of steam. The boxes are banked with earth after placing, and steaming may be done by means of a ¾-in. pipe about 6 ft. long provided with an iron crossbar for a handle. To the end of this pipe a steam hose is attached, connected with a portable. boiler. The pipe is pushed vertically down through one of the holes in the top of the box and, as the steam thaws the ground, it is gradually worked down until it has penetrated the frozen layer. The pipe is then pulled up, the hole blocked and the operation repeated through other holes. With the development of the modern gasoline-operated portable air compressor and the use of air picking, hammering and excavating tools, pavements and frozen ground can be removed much more expeditiously than formerly when chiefly hand labor was employed and thawing has become less of a necessity.

225. Hand Excavation.—The development of excavating machinery and the high prices of common labor have resulted in doing away with much of the pick and shovel excavation formerly employed, so that at the present time hand excavation methods are used mainly on small scattered work, in bottom excavations for the preparation of foundations, or in locations where there is not sufficient room above the ground surface for operating an excavating machine. In locations where many underground structures exist, as in city streets, hand excavation may prove necessary.

[1] This is due to the absence of a protective sod or snow covering, as well as to compactness of the surface material.

It is desirable to throw on one side all material other than rock, excavated from the trench. That from the first 6 ft. of depth should be cast as far to the side as it can be thrown, so as to leave room nearer the edge of the trench for that from greater depths. Unless the material is plastic or sticky, it is usually possible for a man to throw it from the trench until a depth of about 8 ft. is reached. For deeper trenches platforms are constructed about 6 ft. from the surface, supported on the cross braces if the trench is sheeted, or on temporary braces otherwise. Such a platform is made long enough to receive material from a laborer at either end and to provide room for one man to keep the platform clear. Additional platforms will be required for about each 6 ft. of depth, and the material will be cast from the lower to the higher and thence out of the trench These platforms are shown in Figs. 117 and 118. In deep trenches it is necessary to provide a man at the surface to throw the material back from the edge of the trench.

Where trenching machinery is used for pipe sewers, hand excavation must be employed in the shaping of the trench bottom to receive the pipe. In such cases, if the material is firm, the trench may be shaped to the lower half of the pipe, with additional excavations for the bells.

226. Trenching Machinery.—Most of the excavating machinery now in use on sewer work is of the type which excavates and conveys the material to one side of the trench. Formerly in city work, and to a less extent today, trenching machines were used for hoisting the material from the trench after excavation by hand and for conveying it either to a spoil bank or back along the trench to backfill over the sewer.

Trenching machines were first used where the soil was particularly favorable, and close sheeting and heavy bracing were unnecessary. They are now used in more difficult situations both as to soil and obstructions. Pipes across trenches interfere with their use, but in many cases it may be more economical to run the risk of occasional damage to pipes than to resort to hand excavation.

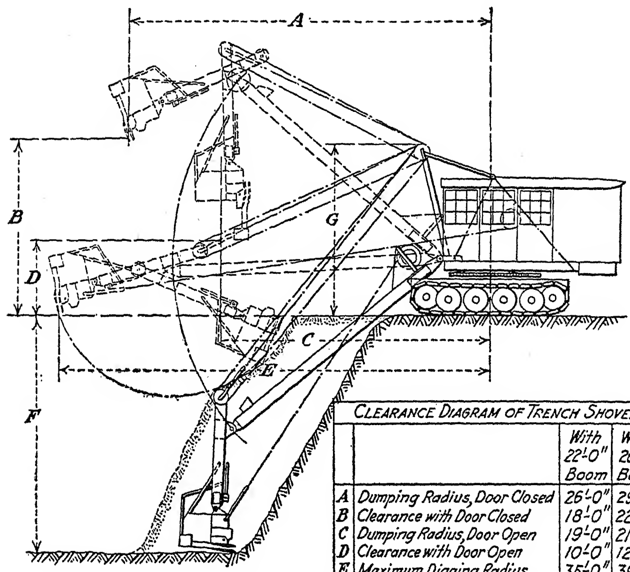
Ditchers or trenchers are usually gasoline driven with caterpillar or wheeled traction. In the *ladder type* of machine, digging is done by buckets carried by an endless chain running over sprockets mounted on the end of an arm suspended from the rear of the machine, which can be adjusted to permit of excavat-

ing to a given depth. The width of trench is governed by the width of buckets selected, each machine being equipped with buckets of several widths. Side cutters may be used for further widening of the trench. The excavated material is carried by an endless belt conveyor operating at right angles to the ditch, and discharged in a windrow at the side of the trench. This type of trench machine is particularly useful where the soil does not

FIG. 109.—Ladder type excavator. (*Austin.*)

require bracing or sheeting and where there is little hardpan. It can also be used where the banks will stand sufficiently long after the machine has passed to enable sheeting and bracing to be placed. Where the banks cave rapidly after the passage of the machine, large quantities of material may fall into the trench and require removal by hand. Fig. 109 shows a trench excavator of this type.

In caving soils the *trench hoe* or trench shovel, or the *drag line excavator* may be used to advantage, since these machines are able to reach back along the trench to remove such material as may fall in before the trench walls are braced. For large sewer



	CLEARANCE DIAGRAM OF TRENCH SHOVEL	With 22'-0" Boom	With 26'-0" Boom
A	Dumping Radius, Door Closed	26'-0"	29'-6"
B	Clearance with Door Closed	18'-0"	22'-0"
C	Dumping Radius, Door Open	19'-0"	21'-0"
D	Clearance with Door Open	10'-0"	12'-0"
E	Maximum Digging Radius	35'-0"	39'-0"
F	Maximum Depth below Level	20'-0"	23'-0"
G	Top of "A" Frame	15'-0"	17'-0"

Fig. 110.—A trench shovel showing method of operating. (Link-Belt.)

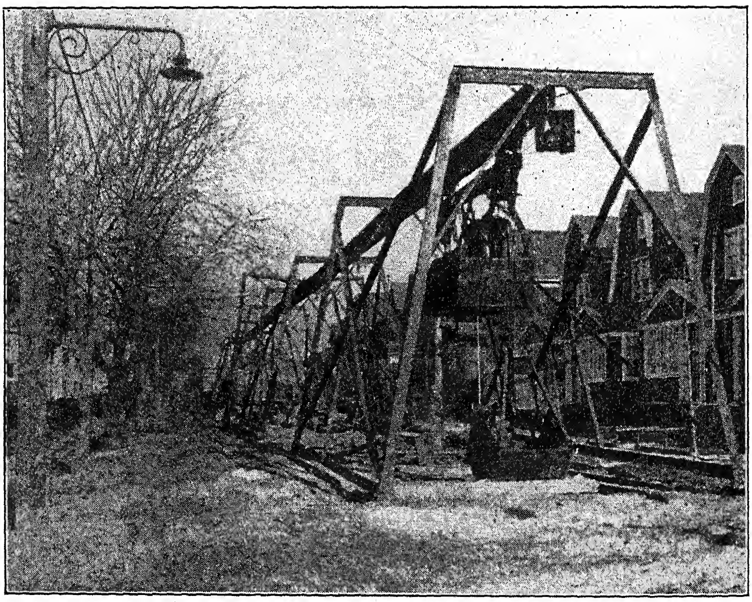


Fig. 111.—Conveyor type excavator.

work trench shovels are advantageous since they can load the surplus material from excavation into trucks for hauling to spoil banks. In Fig. 110 a trench-shovel crawler is shown and the method of its operation suggested by the different positions of the bucket shown.

Where streets are narrow or where for other reasons there is insufficient room alongside the trench to place the excavated material, a *conveyor type excavator* may be used. Fig. 111 illustrates this type of excavating equipment. The ditch is excavated and the material carried by the overhead conveyor back along the ditch where it is placed in backfill. The same conveyor shifts the sheeting and bracing materials along the trench, pulls the sheeting, lays the pipe and handles other material that requires transportation along the trench, by means of a hook at the further end of the car carrying the hoisting mechansim.

227. Rock Excavation.—The method to be adopted for rock excavation in sewer trenches depends upon the quality and quantity of rock to be excavated. Ledge rock varies greatly in character, from that which may be removed by the use of pick and shovel, to hard rock which requires drilling and blasting. Shale and sandstone or rock which is considerably disintegrated and seamy may be removed economically with air picks and shovels.

Where drilling of ledge rock is required, such work is now commonly done with power tools, usually driven by compressed air. The economical removal of ledge from excavations requires a crew experienced in the different processes of the operations. It should include drill operators, a man particularly experienced in the handling of explosives for loading and firing the holes, and be in charge of a foreman who understands not only the drilling but also the blasting operations.

For small holes or on small jobs, the drilling may be done by hand using a drill and hammer or a churn drill. The latter is lifted and dropped, being rotated slightly between blows. The wedge-shaped bit on the lower end of the drill varies in sharpness with the hardness of the rock, being sharper for the softer rock. The bit flares so as to cut a hole larger than the drill, so the latter will not bind in the hole.

The location and spacing of drill holes varies with the character of the rock to be excavated. The amount of material moved by

each shot varies with these factors, the depth of hole, the amount and quality of the explosive used and whether the blasting is in trench or in the open.

Drilling may be accelerated by pouring water into the hole, the sludge thus formed which is not thrown out by the movement of the drill being removed at intervals with a "spoon" or scraper.

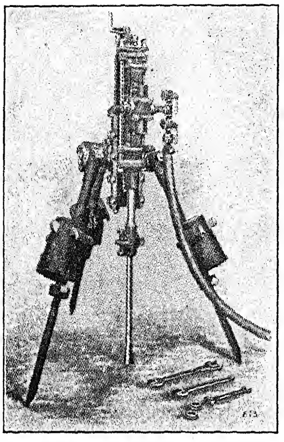


FIG. 112.—The "Sergeant" drill. (*Ingersoll-Rand Company.*)

When the depth of holes, hardness of rock or quantity of ledge to be excavated is such that the economical limitations of hand drilling are exceeded, drills operated by compressed air or steam may be used. The rotary or core drill is not used on sewer work, except rarely for exploration purposes. The rotating bit of this drill cuts an annular hole leaving a core within the hollow drill, which may be withdrawn in convenient lengths and examined as a sample of the material encountered.

The reciprocating drill used on heavy work consists of an air or steam cylinder in which a piston attached to the upper end of the drill shank is moved up and down, rotating the drill

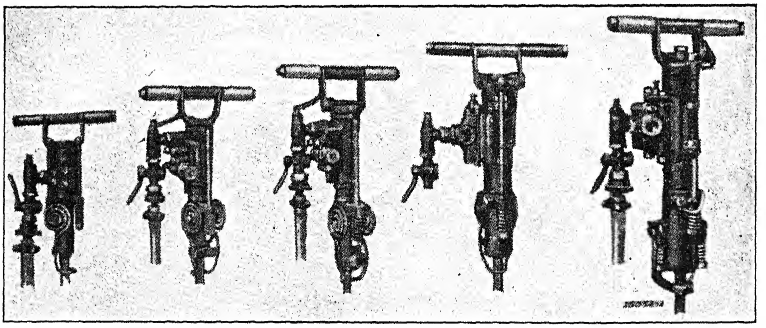


FIG. 113.—The "Jackhamer" rock drill. (*Ingersoll Rand Co.*)

slightly between blows. The drill is usually mounted on a heavy tripod (Fig. 112) which makes the striking of heavy blows possible. This type is rarely used for trench work at the present time. The jack-hammer drill shown in Fig. 113 is a

light, portable drill of the percussion type, usually operated by compressed air. It is readily handled by one man and requires no tripod. It is particularly serviceable in narrow trenches and may be used for drilling holes in any direction. With a portable air compressor the jack-hammer drill is a flexible, easily transported and convenient unit where the work is not too heavy or the holes too deep.

228. Explosives.—An explosive is a substance which, when ignited or detonated by a sudden shock, generates large volumes of gases at high temperatures. The strength and shattering effect of the explosion varies in proportion to the volume of the gas and the suddenness of its formation. Gunpowder is slower in its action than is dynamite, for example, and should be used for softer materials or where a shattering effect is not desired.

Dynamite is the explosive now most commonly used on sewer excavation, although gunpowder or black powder, contractor's powder and TNT are sometimes used.

The active and determining ingredient of dynamite is nitroglycerine. It burns quietly when ignited in the open air, explodes at a temperature of 388°F., and freezes at 41°F. Dynamite which is frozen or chilled cannot be detonated easily, though it is exceedingly sensitive to friction, and sticks should not be broken or cut while in this condition. Thawing can be accomplished in special kettles or thawing boxes, but the operation requires special precautions to prevent explosion.

Nitroglycerine is a very unstable compound, exploding very easily when subjected to detonation or percussion. Being a liquid it is readily absorbed by various substances such as sawdust and wood pulp and these are therefore used as carriers. When its fumes are inhaled or absorbed through the pores of the body, it causes headaches and sickness. True dynamite is a mechanical mixture of nitroglycerine and an inert absorbent. False dynamites are composed of nitroglycerine and absorbent mixtures which themselves enter into the chemical reaction causing the explosion and add to the strength and power which would be furnished by the nitroglycerine alone.

There are a number of other explosives, such as picric acid, blasting gelatine and ammonia dynamite, which have their special uses. Explosives vary in their ease and safety of handling, effect of moisture on their use, compactness, and strength and rapidity of action. Their use should be entrusted only to those

experienced in blasting operations; such men are sometimes required to pass an examination to determine their fitness before being licensed to use explosives.

229. Pumping.—If good cement mortar joints in a pipe sewer are to be obtained, it is necessary to have a dry trench. Cement joints must not be submerged or exposed to running water until the cement has acquired its final set. Removal of water from wet trenches is therefore necessary.

Diaphragm pumps are most often used for unwatering sewer trenches. Such pumps are made in several sizes and are usually power-operated, although hand-operated pumps are sometimes

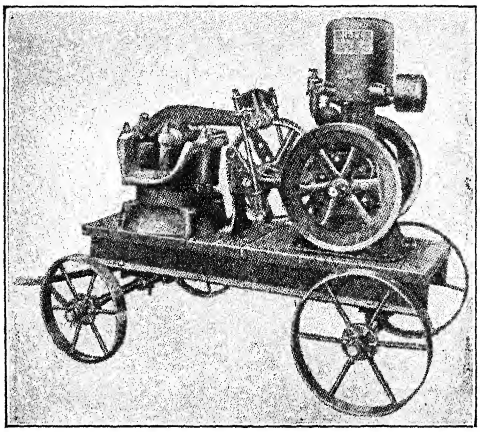


FIG. 114.—Diaphragm pumping outfit truck mounted. (*Novo Engine Co.*)

employed. Good portable gasoline-operated units mounted upon wheels are also available. Fig. 114 shows a portable unit of this type.

When the quantity of water to be handled exceeds the capacity of a diaphragm pump, a centrifugal or a steam vacuum pump is generally employed.

Where the ground is unwatered by means of well points driven into the ground outside the trench, these are connected to a common header and pumping is usually done by means of a centrifugal pump. Fig. 115 shows a sewer trench with well-points connected by suction hose to a 4-in. header through which the water is drawn by pumps situated at intervals along the bank, each caring for a definite number of well points. One of the pump locations can be seen through the suspended sewer section.

Vertical sheeting is seen in the foreground with occasional heavy braces at a higher elevation which support the 4-in. headers. The crane used for handling the sewer sections is used also with a clamshell or drag bucket for excavating the trench. The chimneys used for house-sewer connections are shown, with a rectangular concrete manhole in the background.

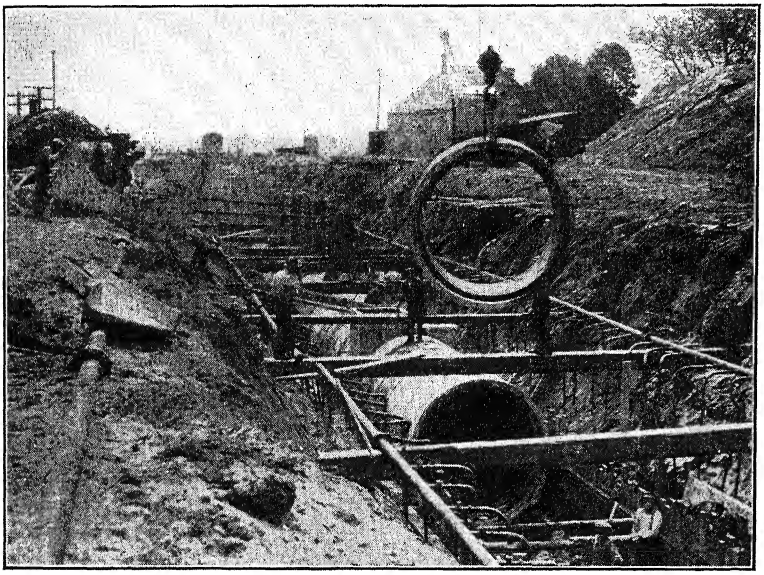


FIG. 115.—Sewer trench unwatered by well-points and pumps.

230. Underdrains.—Underdrains are usually constructed of bell and spigot pipe laid with open joints and are of two classes; construction underdrains, put in to facilitate the building of a sewer and generally abandoned when the sewer has been built, and permanent underdrains, provided with outlets for the purpose of lowering the ground water and thus maintaining a low infiltration to the sewers.

Underdrains in gravel trenches give little trouble from filling. Extreme care must be taken, however, in laying underdrains in fine water-bearing sand. Here the pipes should be thoroughly surrounded with gravel and coarse sand, the coarser materials next the pipe. The joints are sometimes wrapped with one thickness of cotton cheese cloth in strips 6 in. wide, securely tied.

Unless such precautions are followed, the underdrains may become completely stopped with sand.

If an underdrain is properly constructed, little of the sand which causes trouble enters along its length, but only at the upper or free end. It is difficult to prevent this, as the work of excavation keeps the sand stirred and the water carries it into the pipe. In such cases it is customary to keep a rope with a chain attached threaded through the underdrain, which may be pulled back and forth to free it of sand.

SHEETING AND BRACING OF TRENCHES

Trenches may be excavated in most materials to a depth of 3 to 6 ft. without much danger from caving banks. In some materials, as hardpan, excavations may be carried down to greater depths, 12 to 20 ft., without sheeting and without much danger if the trench is not kept open long. Usually, however, it is necessary to shore the sides of trenches thoroughly for the protection of the workmen and to make progress possible.

231. Necessity for Shoring Trenches.—The necessity for shoring arises from the tendency of soils to cave. This caving results usually from one of three causes. First, as the trench wall exposed to the atmosphere gradually loses its moisture content through evaporation, shrinkage cracks are formed. Rain water entering these brings about local swelling which may disrupt the mass and cause caving. Second, the shearing resistance of soils varies within wide limits, and the depth of trench required to produce caving varies correspondingly. When the depth becomes excessive, the soil fails by shear and this failure occurs along a curved rather than a plane surface. Third, where ground water seeps into the trench, a quicksand effect may result if the velocity of flow is sufficient to wash out the finer particles and thus undercut the trench wall. In these three ways caving may be produced and shoring is needed to prevent it. It is well, also, to have a trench so protected as to be safe against damage by storms, which may cut away the sides unless they are well shored; or against failure from other causes. Under some conditions it is desirable to place low banks of earth along the sides and across the ends of the trench to prevent surface water from flowing into it and eroding the earth behind the sheeting.

232. Methods of Shoring.—The method of shoring trenches usually has been left to the judgment of the foreman in charge of the work. Good judgment based on wide experience is essential to the best solution of shoring problems. During recent years, however, the behavior of soils has been investigated more thoroughly and tests have been devised to assist in determining their behavior under the varying conditions encountered in practice. These investigations have clarified considerably the physical principles underlying soil characteristics, and the soils laboratory is coming into use to assist in predicting difficulties to be encountered in underground construction and in devising methods for surmounting them. Soil tests assist but do not supplant engineering judgment based upon experience in these matters, and the young engineer will do well to avail himself of all opportunities to observe the behavior of materials encountered in trench construction and the methods of bracing used.

Trenches are shored by placing braces across them to support the sidewalls in place. This often involves sheathing the sides with planks, called "sheeting" when so used, supported by longitudinal timbers called "rangers" or "wales," which are held by "cross-braces." Sometimes the sheeting is used vertically, sometimes it is placed horizontally and is then often called "box sheeting," and sometimes special forms of shoring are employed which will be described later.

While trenches in rock may often be excavated safely without shoring, it should be kept in mind that rock is treacherous material if seamy and the seams lie at an angle with the horizontal, exceeding the angle whose tangent equals the friction factor for stone against stone. This is particularly true when the seams contain clay or other material which may reduce the angle of repose to a negligible amount. It is often difficult to determine how far back from the face of the trench the rock has been cracked in blasting, and cracks are likely to permit large masses to slide into an unshored trench. When the banks are very treacherous it may be wise to provide close sheeting, but usually where the trenches are of moderate depth frequent stay-braces, or skeleton sheeting, described later, will be sufficient. Blasting makes it troublesome to maintain shoring, and good judgment is therefore necessary in directing the blasting.

The methods ordinarily used for calculating earth pressures disregard the cohesion of the particles of soil. It is this property,

together with the so-called capillary pressure, which sometimes make it possible to excavate a trench to a considerable depth without shoring. A given soil will often show different cohesive qualities under different pressures, though the friction factor remains constant. The cohesion will increase with the increase in depth of the excavation, but an increase in the water content of the soil is likely to be encountered as the trench is carried down, which may affect the capillary pressure. Soils are rarely of uniform character from the top to the bottom of the trench, and wide variations in characteristics may occur.

The adhesive qualities of soil are usually small. If a trench has at some previous time been excavated parallel and near to a new trench, the soil, if allowed to yield, will usually break away to the line of the first trench, showing the lack of adhesion between the original soil and the refilled material. It is often necessary to sheet trenches solely because at some previous time the soil near them has been excavated, the cohesion broken and no adhesion established since then. For this reason the bracing and shoring of trenches in virgin soil is often much simpler than the work needed for similar trenches in city streets which have been excavated frequently.

The time element is important in shoring. Usually the sides of an excavation will be found to exert no marked lateral pressure when first cut. Gradually, sometimes quite rapidly, the soil changes in character, ravels or flakes, and cracks appear in the surface. Sudden and great changes may be caused by rains. Leakage from water mains may soften the material. Voids behind the sheeting may cause a gradual loosening of the soil under the vibration of passing vehicles or when heavily loaded with excavated material. Movements of the earth due to any of these causes or the gradual squeezing out of water, result in changed and often greatly increased pressures on the sheeting, and may also change the point of application of the resultant earth pressure upon it.

233. Behavior of Different Materials in Excavation.—The difference in behavior of the various soils encountered in excavation is due to differences in size and shape of the soil particles and of the moisture content. The differences in cohesion, compressibility, friction factor, capillary pressure and other characteristics, arise from variations in these three factors. Capillary pressure assists cohesion in holding trench walls in

position. This pressure is a summation of the surface tension acting in all of the water surfaces in the partially filled soil interstices, and increases as the soil grains diminish in size. With clay the capillary pressure may be considerable, while with coarse grained materials such as sand it may be small. If the voids are filled with water, as when the soil is saturated, there are no water surfaces and, therefore, no capillary pressure exists. In trench walls, evaporation removes some of this water and capillary pressure assists cohesion in preventing caving. For those trench conditions where this force is a maximum little or no shoring is required, while for saturated clay or quicksand, in which the capillary pressure is zero, the maximum difficulty from soil movement is encountered.

For soils to consolidate, it is necessary that the water be squeezed out. In deep tunnels in clay, the loss of this water by evaporation as it is squeezed out has been known to considerably increase the consolidation and therefore the pressure on shoring.

Little or no shoring may be required in trenches in hardpan. When it caves it is usually in large masses, so that an occasional brace is enough to hold it in place.

While new clay banks stand fairly well for a time after excavation, this material may swell and in many cases water will gradually seep toward the trench, which makes it such a treacherous material that it is desirable to provide close sheeting and heavy timbers.

In dry alluvium sheeting will generally be required, although the banks usually stand well for a time. Occasionally it may be possible to leave spaces between the planks, for the material is not likely to ravel.

Close sheeting is almost always required in trenches in gravel because of the tendency of this material to ravel and roll through small openings. It is usually necessary to shore trenches in gravel when they are more than 3 or 4 ft. in depth.

Sand behaves in different ways, depending upon its size, uniformity and moisture content. Close sheeting is generally necessary and should be placed as soon as the trench is down 3 to 5 ft. Where the sand is fine and contains considerable water, high pressures are likely to be exerted against the shoring, approaching and perhaps exceeding a corresponding hydrostatic pressure. In such material the sheeting must be close, strongly braced and driven a considerable distance below the bottom of the

trench, to help hold the planks in their proper position below the lowest ranger, as well as to prevent the sand in the trench from

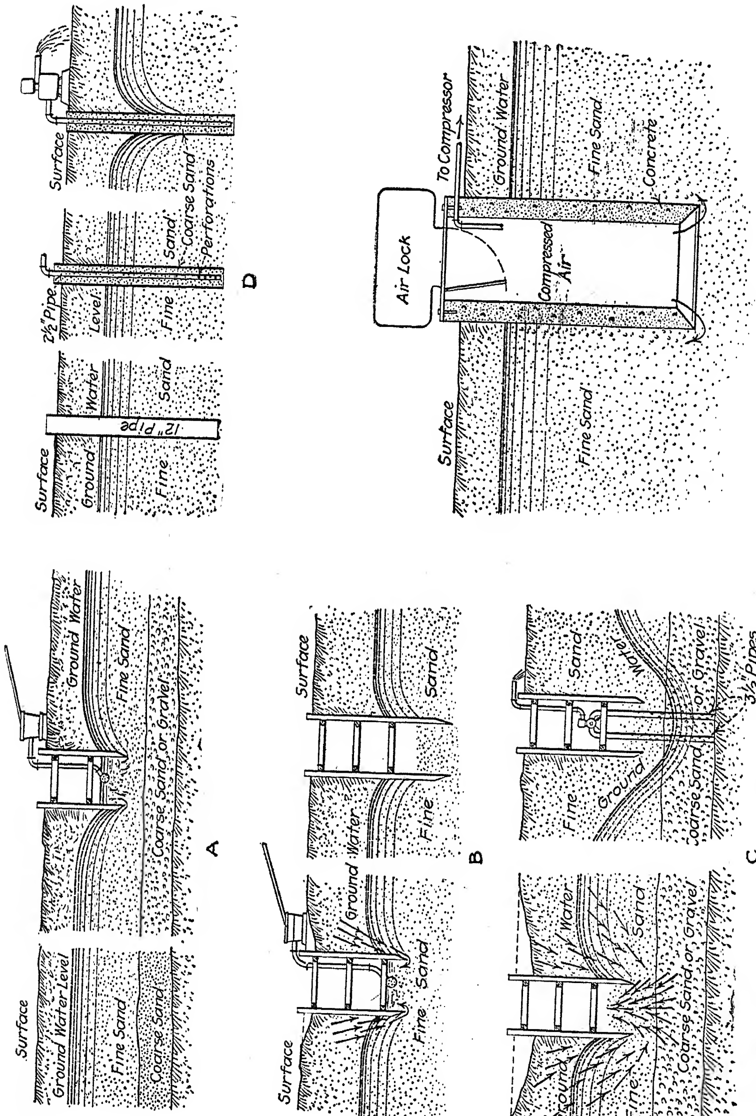


FIG. 116.—Methods of dealing with quicksand.

being forced up by the pressure of the soil outside the sheeting. This also gives the ground water a longer path of travel and reduces its velocity, as it seeps into the trench, below that

required to move the smallest soil particle. If this velocity is exceeded the soil will "flow" into the trench.

The intelligent handling of quicksand is a highly special problem. Fig. 116 illustrates methods of dealing with quicksand.[1]

234. Types of Sheeting.—The sheeting used on sewer work may be of wood or steel. Wood sheeting is in more common use and the different methods of construction may be described as vertical sheeting, box sheeting, poling boards, stay bracing and skeleton sheeting.

235. Vertical Sheeting, although usually more expensive where other methods can be adopted with equal safety, is the

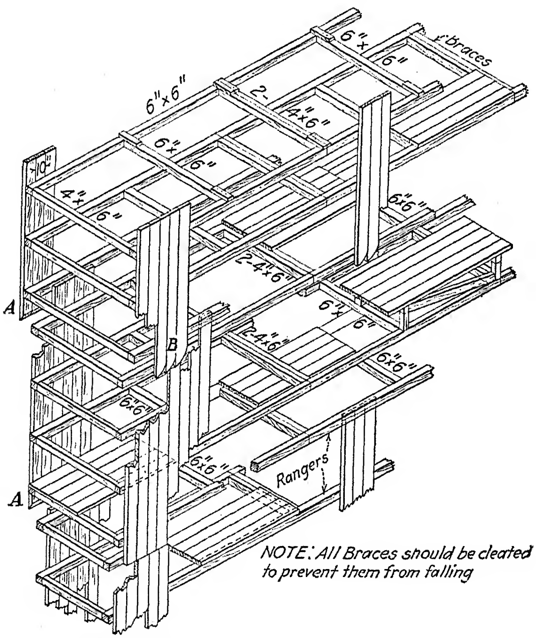


FIG. 117.—Isometric sketch of typical vertical sheeting.

type most commonly used on sewer work and particularly on deep and difficult work, and in trenches in dry running sands and gravels, and in quicksand. Where trenches are excavated under conditions making it important to prevent settlement of pipes

[1] Quicksand, Its Nature, Behavior and Methods of Control. GOW, CHARLES R.: *Jour.*, N. E. Water Works Assoc. 1920; **34,** 171. Fig. 116 has been reproduced from this paper by courtesy of the New England Water Works Association.

and other underground structures, pavements, and abutting buildings, it is often unwise to attempt to use other methods of shoring.

The general method of arranging vertical sheeting is illustrated by Figs. 117 and 118, which are companion drawings of a narrow trench carried to such depth as to require three sets of sheeting. The cross-timbers or "braces" are shown cleated, which is good practice upon work of this kind. Staging platforms upon which

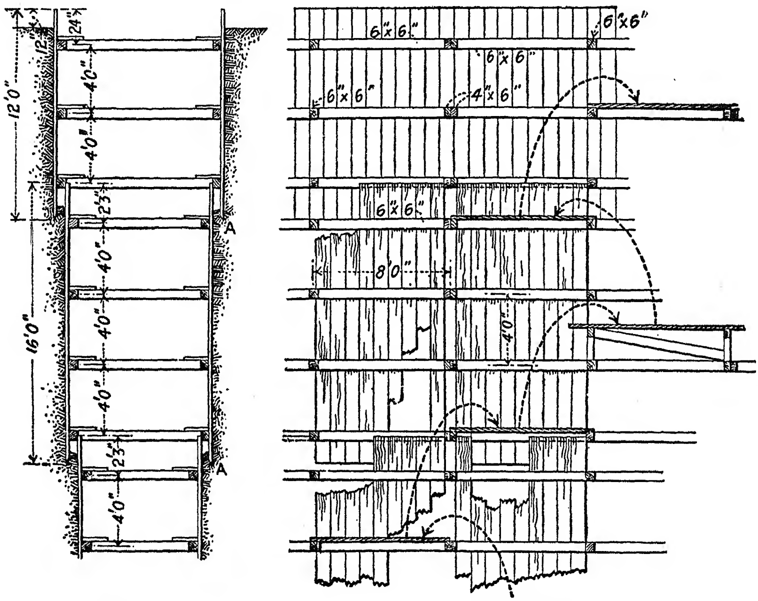


FIG. 118.—Typical trench sheeting.

the excavated material is thrown, are shown extending half way and all the way across the trench, as both methods are commonly used. The general arrangement of braces, rangers and sheeting is illustrated but the dimensions given should be varied according to conditions.

The common practice is to use sheeting planks sharpened with "chisel" edges, placed with the beveled side toward the inside of the trench as shown at *A*. In some cases, especially in difficult work where it is necessary to drive the sheeting long distances ahead of the excavation, the planks may be

beveled in two directions, as shown at *B*, Fig. 117, the object being that the planks shall drive straight and shall tend to hold tightly against the planks previously driven.

236. Box Sheeting.—Horizontal or "box" sheeting is sometimes used where the material excavated is such that the banks will stand untimbered for a considerable length of time without great danger of their caving. With it less care need be taken in trimming the banks of the trench plumb than in the case of vertical sheeting, as it is not absolutely necessary, though it is preferable, that each plank be placed directly under the one above. After the excavation has been carried as deep as practicable without bracing (usually the width of three or four planks), the planks are placed along the sides of the trench and held in position by 2-in. cleats placed vertically at each end and usually in the center. These cleats are in turn held firmly by cross-braces. As the width of the trench between the opposite planks is likely to vary, screw braces are to be preferred to the ordinary driven brace, as they are more readily adjusted and by their use the trench may be more quickly braced. The screw braces are also preferable, as there is little or no jar in connection with placing them, while if driven braces are used the jar resulting from the driving may be sufficient to endanger the banks.

Box sheeting has been used in some cases where the material excavated was very unstable, such as coarse gravel and dry sand. In such cases the excavation can be carried only deep enough to accommodate a single plank at a time. Each set of planks is temporarily braced until the trench has been carried deep enough to accommodate uprights supporting three or four planks, after which the uprights, with one or two braces against each pair, may be substituted for the individual plank braces. In this class of material box sheeting may be more expensive and troublesome than vertical sheeting, but it is quite useful under certain difficult conditions, as for example in trenches between car tracks or under structures which interfere with the placing of vertical sheeting. The use of the wooden horses and temporary stages required by plank drivers is also avoided by the use of box sheeting, which is often advantageous, especially in narrow streets.

Fig. 119 shows a trench about 12 ft. in depth, the banks of which were supported by two sets of box sheeting, each three planks in depth. The planks used were 2 in. thick, 12 in. wide,

and 18 ft. in length. The cleats were 2 by 12 in. and 3 ft. in length. Extension braces were used, as shown in the sketch. The material excavated was a dry alluvial clay.

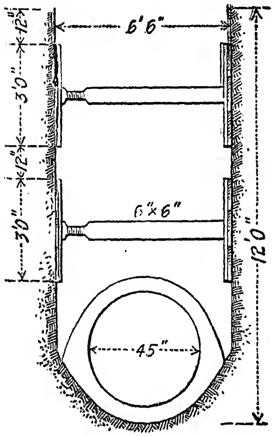


Fig. 119.—Box sheeting.

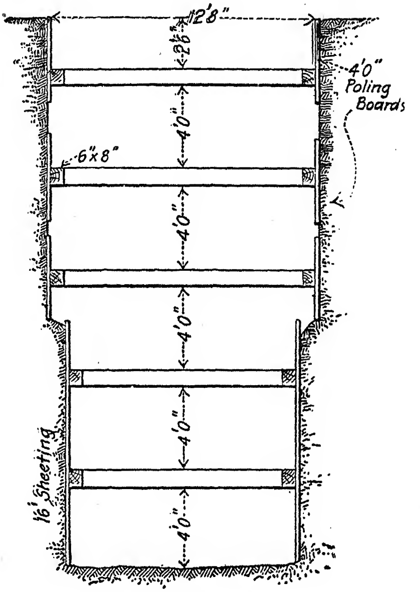


Fig. 120.—Poling boards and vertical sheeting.

237. Poling Boards.—This method of timbering is similar to that of vertical sheeting, except that short planks 3 or 4 ft. in length, called "poling boards," are used and the sheeting is placed against the banks from time to time as the excavating proceeds instead of being driven. The planks are placed vertically against the properly trimmed banks, and are supported by means of rangers and braces, as in the case of vertical sheeting. In some cases where the banks stand well it is possible to get along with a single set of rangers and braces for one set of poling boards, as illustrated in Fig. 120. This, however, is not as safe as where two sets of rangers and braces are used for each set of poling boards.

The chief disadvantage in the use of poling boards lies in the fact that if there is a collapse in the timbering of any portion of the excavated section, resulting from the running out of the material behind the sheeting, serious deformation or even collapse of the

entire timbering of the section may result. The danger of this is much greater with poling boards than with vertical sheeting.

238. Stay Bracing.—When the excavation is in hardpan, dry clay or dry loam, it is often unnecessary to provide close sheeting, although some support for the banks may be needed. In such cases stay bracing, or skeleton sheeting, will prove economical. The simplest form of stay bracing involves the placing of pairs of vertical planks braced tightly against the banks of the trench at intervals of 6 to 10 ft. The braces may conveniently serve as

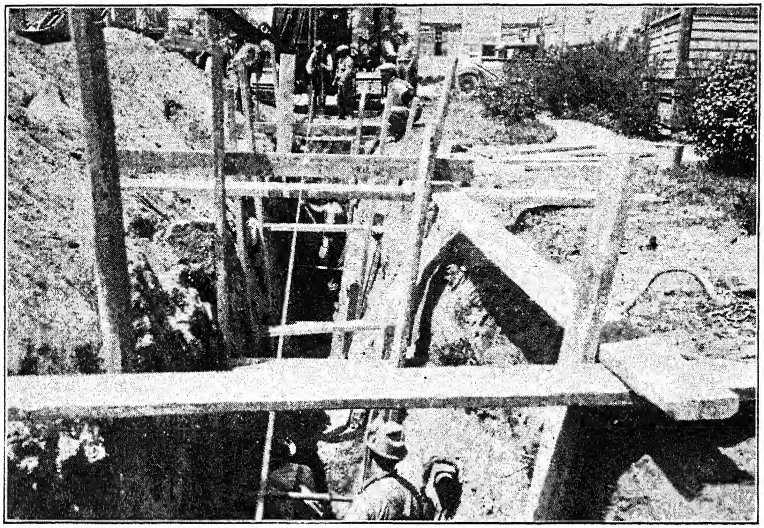


Fig. 121.—Stay bracing.

supports for staging platforms in trenches which are sufficiently deep to require that the material be staged out. If a trench must be left open for several days, the use of stay bracing is not to be recommended unless conditions are especially favorable, as heavy rains are likely to soften the banks and cause expensive and dangerous caving. Fig. 121 shows the method of supporting the banks of a trench by means of stay braces.

239. Skeleton Sheeting.—In some cases where it is difficult to determine prior to the completion of the excavation, whether it will be necessary to use vertical sheeting, skeleton sheeting is used. This consists of the usual rangers and braces with planks between the rangers and the banks of the trench opposite

the braces. By putting in these frames, it is possible to slip planks in later without delay wherever found necessary. It is often desirable to place such planks at intervals of 2 or 3 ft. where it is unnecessary to provide close sheeting.

240. Jetting.—Where sheeting must be driven ahead of excavation, occasionally it is found impossible to make substantial progress in the driving of sheeting by ordinary methods. In such cases a jet of water may be forced into the soil just below the bottom of the plank so that as the soil becomes softened the plank can be driven with comparative ease.

In some cases the bottom of each plank is fitted with a special appliance by means of which the jet of water is introduced into the soil immediately below the plank. In other cases the water may be conveyed to the lower end of the sheeting by a $\frac{3}{4}$- or 1-in. pipe, independent of the sheeting itself, and pushed down by hand. Where the planks are fitted with special jetting shoes, it is economical to drive them a considerable distance below the bottom of the trench at one operation.

241. Practical Suggestions on Timbering.—Good workmanship is essential to the successful timbering of trenches.

The banks should be carefully trimmed to a plane surface before placing the sheeting.

In excavating a trench in loose material, hay or meadow grass will be found useful to prevent the material from running through the cracks in the sheeting.

The banks of a trench should never be allowed to move, if it is possible to prevent movement, as this destroys cohesion and increases the earth pressure against the sheeting. The sheeting and bracing should be placed in the trench as soon as practicable, in order to prevent any movement of the top layers of the soil. The sheeting should be driven as fast as the excavation proceeds and in quicksand it is, of course, necessary to drive the sheeting in advance of the excavation. Great care should be taken to prevent the banks from raveling where notches or "windows" are required. The pressures exerted upon timbering increase greatly as the soil back of the sheeting becomes cracked, loosened, or in any way shifts its position. Such movements of the banks are likely to cause the breakage of adjacent water pipes and sewers, resulting in the partial or complete saturation of the soil with the water leaking from them, and producing pressures approaching hydrostatic pressure.

Braces should always be tight. Where they are short and of wood they may be cut to correct length, and driven, but where they are long they should be tightened by hard wood wedges driven to refusal. Upon the lighter work extensible screw braces will be found more convenient than wooden braces.

Except upon heavy and expensive work it will generally be found more convenient to use rangers and braces of the same size from the top to the bottom of the excavation, than to attempt to vary their size in accordance with theory. If, as is frequently the case, it seems desirable to increase the strength of the timbering toward the bottom of the excavation, or at any other point, this can be done by decreasing the space between the rangers, and adding braces.

242. Materials and Tools.—Ordinarily the first cost of timber is an important if not the chief consideration in its selection. If sheeting requires hard driving, it is advisable to get either a soft, tough wood or a hard, strong wood. Spruce is very satisfactory and oak stands driving fairly well. Hemlock is usually unsatisfactory because it splinters. Oak is heavy and likely to warp.

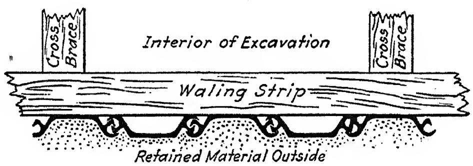


Fig. 122.—Typical section of wall of Lackawanna arched web steel sheet piling showing interior bracing.

Sheeting planks should not be more than 10 in. wide, except notched planks, nor less than 6 in. The tops of the sheeting planks should be dressed to receive plank caps, placed over them to protect the ends against brooming under the driving maul. The cast iron caps should fit loosely on the plank ends or they will be cracked in driving.

Steel sheet piling is used extensively for heavy engineering foundation work, but rarely for trench work. It is not as easily handled, driven or pulled as plank, but its useful life is much longer, and it has other special advantages for cofferdams or large and difficult foundations.

Fig. 122 illustrates the method of bracing used with steel sheet piling.

The 16-lb. hammer is most useful for driving cross-braces and permits a blow to be delivered close to the end of a brace more effectively than by a sledge hammer. A lighter hammer is undesirable and a heavier one is clumsy for ordinary work.

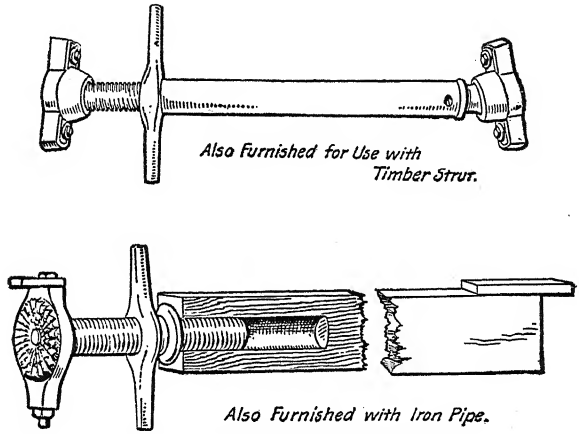


FIG. 123.—Extensible trench braces.

Extension braces, Fig. 123, are often used in light trench work, being more quickly applied than timber struts and requiring no driving, an advantage where banks are unstable. These

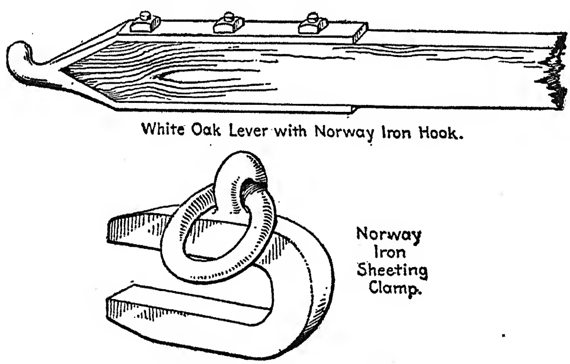


FIG. 124.—Sheeting or plank clamp and lever.

braces may be provided with lugs on both ends, which overhang the rangers and prevent the braces from falling. The extension brace is usually fitted with lap-welded iron pipe, and its length may be varied by substituting different lengths of pipe; sometimes

timber braces with screws on one end are used, in which case one end of the strut is bored out to receive the screw.

Sheeting planks are pulled by means of a wrought-iron clamp slipped over them and raised by a long timber lever with a wrought-iron hook at the end, shown in Fig. 124.

243. Shoring Tunnels.—Tunnelling as a method for constructing large sewers is occasionally used. The conditions under which it would be justifiable include excessive depth of sewer, traffic congestion, presence of surface structures which

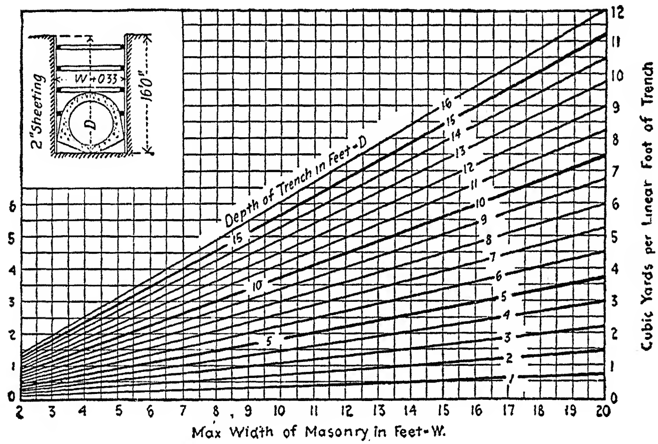


FIG. 125.—Theoretical quantity of excavation for trenches 1 to 16 ft. deep, for sewers of 2 to 20 ft. outside diameter.

interfere with the open cut method, and the existence of sub-soil conditions favorable to tunnelling. Shoring is a major problem in such work.

In boring the tunnels for the Detroit sewers, a stiff clay subsoil permitted the use of an auger-like borer. The soil, except for a few sand pockets and an occasional boulder, would stand unsupported for some hours for lengths of 12 to 15 ft., which simplified the shoring problem. In driving a tunnel through soft or loose material, special methods of timbering are required, which permit the roof timbers to follow closely or to project ahead of the shield. Since the timbering for tunnels is a special and complicated problem, a detailed discussion is not warranted here, but the student is referred to any standard work on "Tunnelling" for such information.

244. Quantities of Trench Excavation.—The cost of sheeting, bracing and pumping will be about the same whether the trench is of the exact width needed or a foot or two wider. The cost of picking, shoveling, backfilling and hauling surplus material to the dump will be closely proportional to the quantity of material

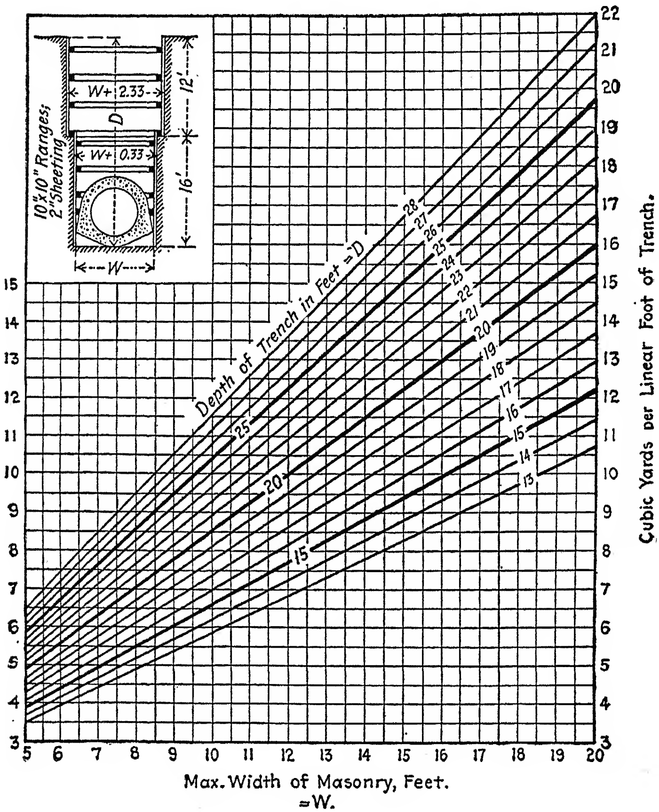


FIG. 126.—Theoretical quantity of excavation for trenches from 13 to 28 ft. deep for sewers of 5 to 20 ft. outside diameter.

actually excavated. Where some types of machinery will be used, the width of the trench may be greater than necessary for the structure. In hardpan and dry clay, the banks may stand without bracing, thus making it possible to dig the trench of practically the width of the structure to be built in it, while the same sewer in a trench in running sand may require heavy

timbering. In rock excavation it is often difficult to excavate exactly to the line theoretically required.

To prevent controversy as to the quantity of excavation to be paid for on contract work, it is customary to prescribe certain limits, called "lines of excavation," to which the quantity to be paid for shall be calculated, but the contractor may, if necessary, excavate beyond these and if the structure can be built in narrower excavation he is not required to excavate the full width shown upon the drawings. Some engineers provide in the contract that earth excavation shall be paid per linear foot of sewer constructed. This is an excellent method, provided the preliminary engineering has been done with sufficient care and thoroughness so that there is no possibility of a change in plans.

The quantity of excavation required for trenches for masonry sewers from 1 to 40 ft. in depth and up to 20 ft. in width may be obtained from Figs. 125 and 126. These diagrams are based upon the assumption that the top set of sheeting will be 11 or 12 ft., the bottom set 16 ft., and the intermediate set 12 ft., that 2-in. sheeting will be used, and that 10-by 10-in. rangers and braces will be employed. The quantities of excavation obtained by the use of these diagrams should be regarded as approximate only, for special calculations will be needed for any case where exact quantities are required. The averages of actual cross-sections of about 25 sewer trenches in Louisville showed that the excess excavation for small sewers having an extreme outside width of 6 ft. was nearly 33 per cent, whereas the excess excavation for sewers from 12 to 16 ft. wide, outside, was 1 to 5 per cent, which indicates the decreasing importance of excess width as the width of the sewer increases.

CHAPTER XI

CHARACTERISTICS AND BEHAVIOR OF SEWAGE

245. General Characteristics of Sewage.—Knowledge of the properties of sewage is essential in the design and operation of both collecting systems and disposal works. General acquaintance with the nature of sewage, as affecting chiefly its hydraulic properties, commonly suffices for the planning of sewers and sewer appurtenances. Design and operation of sewage treatment works and control of the pollution of water courses into which sewage is discharged with or without treatment presuppose, however, a thorough appreciation of the physical, chemical and biological characteristics and behavior of sewage and sewage matters.

From a consideration of its sources, *sewage* may be defined as a combination of the liquid, or water-carried, wastes conducted away from residences, business buildings and institutions, together with those from industrial establishments; and with such ground, surface and storm water as may be present. The liquid, or water-carried, wastes originating in residences, business buildings and institutions are known as *domestic sewage.* Those resulting from the manufacturing processes employed in industrial establishments are called *industrial wastes.*

Although sewage is composed largely of water (99.9 per cent), it contains sufficient quantities of substances which are offensive in character and behavior or dangerous to the public health to make the economic and sanitary disposal of sewage a problem of far-reaching importance.

In appearance, sewage resembles dirty dish water or bath water to which has been added floating matter, such as fecal solids, bits of paper, matches, grease, vegetable debris and fruit skins. *Physically,* sewage contains matter in suspension and matter in solution. Of the suspended solids, some will settle when the transporting power of the water is decreased by a reduction in its velocity and some will remain in suspension even during protracted periods of quiescence. The quantity of material carried can be realized from computations of the bulk of the substances,

other than water, discharged through sewerage systems. The Metropolitan Sewerage Commission of New York, for example, estimated that the city's sewage contained, per thousand population annually, 14 tons of feces, 8 tons of toilet paper and newspaper, 11 tons of soap and washings, 8 tons of street wastes, and 4 tons of miscellaneous substances; a total of 45 tons.

When sewage is discharged without treatment, settling solids are often deposited on the bottoms and sides of streams and form silt and sludge banks which interfere with the regimen of water courses, especially during periods of low flow. Floating solids, fats and oily products may become stranded along the banks or may form unsightly scums which reveal the sewage origin of the floating matters. The layman is apt to object to the appearance of sewage-polluted water rather than to the sanitary hazards.

Chemically, sewage contains substances of animal, vegetable and mineral origin. The animal and vegetable substances, called collectively *organic matter,* are in large part offensive in character or behavior. They are made up of complex chemical substances which are readily broken down by biological and, to a lesser degree, chemical action into other, usually simpler, compounds. Uncontrolled decomposition of this organic matter, which constitutes about 50 per cent of the sewage solids, is commonly accompanied by objectionable odors and other noxious conditions in the streams or other bodies of water into which sewage may be discharged.

Biologically, sewage contains vast numbers of living organisms, among which the bacteria predominate. One gallon of sewage may harbor from 20 to 250 billion bacteria. Most of these organisms are harmless to man and are largely engaged in the beneficent activity of converting the complex organic constituents of sewage into simpler, more stable, organic and mineral compounds. Sewage, however, may contain bacteria or other organisms that have come from persons sick with typhoid fever, dysentery, or other so-called water-borne diseases. Some disease-producing (pathogenic) organisms are commonly present, and it is these that constitute the real danger of sewage to the public health.

246. Concentration, Composition, and Condition of Sewage.—The *concentration, composition* and *condition* of sewage depend upon the quantity, nature and freshness of the sewage matter contained in it.

Concentration is a term commonly used to designate the proportion of sewage matter to water. A strong or concentrated sewage contains a relatively larger proportion of sewage matter, while a weak or dilute sewage contains a relatively smaller one. There is no recognized standard by which a sewage may be classed as strong or weak. In sewage-disposal problems, the organic matter in sewage, being more important than the mineral matter, is usually the basis for classification. The quantity as well as the nature of the organic matter determines the strength of sewage.

Composition is defined by the different constituents of a sewage and their quantities. If the quantities of certain constituents are high as compared with average experience the sewage is called strong, if low the sewage is called weak. Composition must be determined before concentration can be judged.

Condition is a property of sewage which depends upon the changes that have taken place during the passage of the sewage through the sewerage system. As it flows through the collecting and outfall conduits, the coarser solids are broken up by mechanical abrasion, and the organic matter is decomposed and changed in character by chemical and biological action. The longer the sewage flows or stands, the more are its constituents changed; fecal matter and other suspended solids are comminuted; bacterial activity increases enormously and assists in the breaking-down of the complex organic compounds; the oxygen originally dissolved in the water is reduced and may even disappear, so that from a *fresh* condition the sewage becomes *stale* and, when all the oxygen is used up, *septic.*

SEWAGE ANALYSIS

247. Scope of Sewage Analysis.—In general, a sewage analysis is designed to give information upon the presence and properties of the sewage matters. To this end the laboratory analysis may comprehend the results of many separate determinations, which in one way or another measure or reflect the physical, chemical and biological qualities of the sewage.

The examination of sewage is unlike the chemical analysis of many common substances in which the exact quantities of each constituent element or group of elements are determined. Sewage matter is too complex and varied to permit this. It is

therefore, necessary to depend very largely upon tests which measure properties of the sewage constituents rather than their exact make-up. For this reason, too, a variety of more or less interrelated tests are made which taken together rather than separately, may be interpreted. The variety and interrelation of the tests commonly performed is best shown by grouping them in accordance with some scheme as that of Table 63. An important test that is not incorporated in this schedule is the determination of temperature. The recording of temperatures is important in modern sewage treatment methods because most of them are dependent upon biological activity which is stimulated or retarded in accordance with the prevailing temperatures. Other tests, too, are not shown, such as those for sulphates, phenols, poisonous metals, or other substances that may be significant under certain circumstances.

TABLE 63.—APPROXIMATE CLASSIFICATION OF TESTS COMMONLY USED IN SEWAGE ANALYSIS

Gases and volatile constituents	Mineral matter	Organic matter	Mineral and organic matter	Living organisms	
				Bacteria	Plankton
Odor	Fixed solids	Volatile[1] solids	Solids	Bacteria	Animal forms
Dissolved oxygen	1. Total	1. Total	1. Total	1. At 20°C.	Plant forms
Hydrogen sulphide	2. Suspended	2. Suspended	2. Suspended	2. At 37°C.	
	(a) Settling	(a) Settling	(a) Settling		
	(b) Non-settling	(b) Non-settling	(b) Non-settling	B. Coli	
	3. Dissolved	3. Dissolved	3. Dissolved		
	Ammonia[1] *N*	Organic *N*	*H*-ion concentration		
	Nitrite *N*	Albuminoid *N*			
	Nitrate *N*	Oxygen consumed			
	Chlorides	Biochemical oxygen demand			
	Alkalinity				
	Hardness	Relative stability			
	Iron	Ether-soluble matter			

[1] Includes both organic and inorganic substances.

The choice of tests depends upon the use to which the analysis is to be put. Not all the tests given in Table 63 need be made in

every analysis. The analytical technique of performing the various tests cannot be described in this book. The present chapter is devoted merely to a statement of the nature and significance of the tests. The details of analysis, as commonly performed in the United States, are elaborated in "Standard Methods for the Examination of Water and Sewage" published by the American Public Health Association.

248. Expression of Analytical Results.—With the exception of a few tests, such as temperature, odor, hydrogen-ion concentration and living organisms, the results of sewage analyses are expressed as *parts per million* (p.p.m.) by weight, assuming that a liter of sewage as measured for analysis weighs 1 kilogram. Under ordinary circumstances, therefore, parts per million are equivalent to milligrams per liter.

The varying quantities of water in different sewages make it difficult to compare the average analysis of the sewage of one community or even of one sewerage district with that of another. If, however, the quantity of sewage and the number of the contributing persons are known, it is possible to compare the composition of the sewage on the basis of the sewered population. The accepted method of stating composition is then in terms of *grams per capita per day* (gm. per cap. daily). For a sewage flow of 100 gal. per capita daily 1 part per million is equivalent to 0.378 gram per capita per day.

Other methods of statement are sometimes found useful, such as pounds per capita per day (lb. per cap. daily), tons per 1,000 population per year (t. per 1,000 pop. yearly) and pounds per million gallons (lb. per m.g.). One gram per capita per day equals 0.00221 lb. per capita per day and 0.403 ton per 1,000 population per year. One part per million equals 8⅓ lb. per million gallons.

Statement of the results of those tests which cannot be expressed by weight will be discussed later in connection with their interpretation and significance.

249. Average Composition of Sewage.—The average composition of sewages in America and Europe is shown in Table 64. Bacterial figures are not included.

The composition of the sewage of a community changes rapidly from time to time on account of the fluctuating quantities of constituents contributed and the entrance of more or less ground water, surface water and storm water into the sewers. The night

TABLE 64.—COMPARATIVE AVERAGE SEWAGE ANALYSES

Constituents	Parts per million						Grams per capita daily					
	Large American cities (combined sewers)	American manufacturing cities	Small American manufacturing cities	American residential and rural communities	Large English manufacturing cities	German communities of the Emscher district	Large American cities (combined sewers)	American manufacturing cities	Small American manufacturing cities	American residential and rural communities	Large English manufacturing cities	German communities of the Emscher district
Sewage flow, gal. per capita per day	178.0	95.0	69.0	80.0	49.0							
Nitrogen												
Organic nitrogen	8.0	24.1	23.8	18.0	.....	16.9	6.4	8.0	5.8	5.5	.....	11.1[1]
Albuminoid nitrogen	7.0	11.9	11.3	7.8	14.8	......	......	2.7	2.8	2.4	2.9	3.6
Ammonia nitrogen	10.6	26.5	38.9	27.2	36.5	33.7	7.8	8.3	9.5	7.5	6.9	7.5
Nitrite nitrogen	0.11	0.26	.....	.....	......	......	0.05					
Nitrate nitrogen	0.44	1.19	.....	.....	......	......	0.21					
Oxygen consumed	59.0	133.0	107.0	71.0	266.0	......	35.6	48.0	27.0	21.6	49.0	
Biochemical oxygen demand (5 days)	208.0	230.0	283.0	143.0	......	......	140.0	83.0	74.0	54.0		
Chlorides	48.0	109.0	83.0	47.0	162.0	209.0	37.0	44.9	21.0	13.4	30.0	
Alkalinity	153.6	129.0	.....	.....	......	......	161.0					
Solids												
Total	1355.0	1058.0	730.0	603.0	1896.0	2044.0	567.0	263.0	185.0	183.0	362.0	550.0
Volatile	452.8	635.0	448.0	393.0	......	623.0	185.0	147.0	112.0	120.0	.....	151.0
Fixed	902.8	423.0	282.0	210.0	......	1421.0	382.0	116.0	73.0	63.0	.....	399.0
Suspended solids												
Total	214.2	384.0	242.0	342.0	668.0	601.0	127.0	131.0	58.0	105.0	126.0	153.0
Volatile	144.3	288.0	203.0	260.0	......	332.0	72.0	88.0	49.0	82.0	.....	82.0
Fixed	69.3	96.0	39.0	82.0	......	269.0	45.0	43.0	9.0	24.0	.....	71.0
Dissolved solids												
Total	1052.5	608.0	488.0	261.0	1228.0	1443.0	441.0	167.0	126.0	77.0	236.0	397.0
Volatile	241.5	270.0	245.0	133.0	......	291.0	99.0	74.0	63.0	39.0	.....	69.0
Fixed	811.0	338.0	243.0	128.0	......	1152.0	342.0	93.0	63.0	39.0	.....	328.0
Fats	25.3	37.0	.....	.....	......	......	25.0					

NOTE.—German figures are based on flow per capita of total population and are probably somewhat lower than strictly comparable figures.
[1] Total nitrogen.

sewage is much weaker than that of the day, and hourly analyses of sewage show that its composition depends rather directly upon the activities of the population.

Causes similar to those affecting the hourly variations change the quality of the sewage on different days. The greatest departure from the average daily quality occurs on Sunday, when industrial wastes are not discharged into the sewers and household activities are checked. Where the week's washing is done on Monday, the quantity of sewage is increased by the discharge of wash waters, and the soap and other ingredients affect the quality of the sewage.

Sewage flowing during the spring or "wet" months is generally considerably weaker than that of summer and early autumn.

The composition is affected also to some extent by temperature and bacterial action, as well as by dilution. Seasonal changes in food consumption and industrial operations likewise influence the composition of sewage.

250. Sampling.—It will be seen from the statements in the foregoing section, that it is necessary to obtain samples, planned and taken intelligently, in order to obtain from them a correct knowledge of the character of the sewage or wastes. The results of analysis of a single sample usually mean little. It is advisable to secure a series of samples and to be guided by the average results of analyses, giving due consideration to the variations. It is possible, however, to be misled by both average results and analyses of isolated samples, unless the conditions of sampling are known and appreciated.

Sampling of sewage, industrial wastes and effluents from treatment works commonly involves the collection of composite daily or sometimes weekly samples. Individual portions are taken at frequent intervals, half-hourly or hourly, and are mixed at the end of the sampling period or combined in a single container as collected. Whenever possible the individual portions should be combined in volumes proportionate to the rates of sewage flow. Each portion, according to "Standard Methods," should be not less than 120 cc., preferably 1 liter. Proportioning is conveniently done by taking a simple multiple in cubic centimeters of the flow in million gallons daily or some other unit of flow. When the analysis cannot be performed within 4 to 6 hours after collection, a preservative must be added. Chloroform, formaldehyde, or sulphuric acid is used for this purpose. For the

details of preservation, "Standard Methods" should be consulted. Samples for bacteriological analysis or the determination of relative stability or biochemical oxygen demand must be free from preservatives.

COMMON CONSTITUENTS AND PROPERTIES OF SEWAGE

251. Solid Matters in Sewage.—Part of the total solids in sewage come from the water which is its principal constituent. Not all of the water supplied to consumers reaches the sewers. The portion that does is supplemented often by water furnished from private supplies, and usually by ground water and storm water that enter the system. If the water from the different sources is hard or otherwise highly mineralized, it will contribute a considerable proportion of the solids in the sewage. Other solids have their origin in kitchens, laundries and bathrooms or in wash water from other parts of households, hotels, office buildings and institutions. Feces, urine and paper furnish further solid matter, and industrial wastes contribute still more.

TABLE 65.—ROUGH ESTIMATE OF TOTAL SOLIDS IN THE SEVERAL CONSTITUENTS OF SEWAGE

Constituents	Grams per capita per day	
	Items	Total
Water supplies and ground water, assumed to be soft	12.7	
Feces	20.5	
Urine	43.3	
Toilet and news paper (suspended)	20.0	
Sinks, baths, laundries and other sources of domestic wash waters	86.5	
Total for residential sewage from separate sewerage system		183.0
Industrial wastes	200.0	
Total from industrial city with separate sewerage system		383.0
Storm water	25.0	
Total from industrial city with combined sewerage system		408.0

In combined systems street washings and storm waters impose their load of solid matter on the sewage.

The quantities of solids in sewage depend, therefore, upon the character of the water in the sewers, the habits of the population, the nature of the industries and the use of a combined or separate system. As a result there is generally considerable variation in the amount of solids in sewage of different cities, even when they have about the same population. Table 65 shows an estimate of the average quantities of solids in sewage with respect to their origin.

252. Suspended, Colloidal and Dissolved Matter.—The solid matters in sewage are found to exist in three states, as (1) suspended matter, (2) colloidal matter and (3) dissolved matter.

Differentiation of these three states is dependent upon the size of the sewage particles in the surrounding water and can be explained as follows:

If fine sand and water be shaken, there results what is called a *suspension*. The separate particles in suspension may be seen, and in time they will settle out; but the finer the particles, the longer will it take them to deposit. If they are fine enough, it will take an infinitely long time for them to do so. To particles which will not settle out for a very long time the term colloidal matter is applied, and it is applied to combinations with increasing fineness of grain until the particles can no longer be distinguished with the best microscope or by ultra-microscopic methods, as by passing a beam of light in one plane, known as a Tyndall beam, through the liquid and observing the particles as dust particles may be seen in air dancing in a beam of sunlight which penetrates into a darkened room. With decreasing particle size, what are called true solutions are finally reached, such as the solutions of common salt or sugar in water. It is evident that there is perfect continuity from one extreme to the other, and that there are no exact delimitations of the three states.

When observed under the microscope, particles larger than 3μ to 5μ[1] exhibit no motion, whereas particles with diameters of less than 1μ to 2μ manifest what is called *Brownian movement*. This is explained as motion induced by bombardment of the particles by the molecules of the liquid. The smaller the particle, the greater is the amplitude of its motion. The friction induced

[1] μ = 1 micron (plural microns or micra) = 0.001 mm. = $\frac{1}{25,000}$ in.

by this motion is probably responsible for the electrical charges on the surface of the particles, associated especially with particles in the colloidal state.

In the light of these statements, true solution may be conceived of as a state in which the particles are so small and their velocities so great that they diffuse rapidly through the water.

253. Determination of Solid Matters in Sewage.—The quantity of solid matter in sewage is determined by evaporating a known volume of sewage and drying and weighing the residue. This residue is a measure of the *total solids* in the sewage. If this residue is ignited, *i.e.* heated sufficiently to burn off the volatile matter, the weight of the remaining substances gives the so-called *fixed solids* or approximately the mineral matter. The difference in the weights of the total solids and fixed solids is called the *loss on ignition*, and is a measure of the *volatile solids*, or approximately of the organic matter. Ignition gives an idea of the quantity, but not of the character of the organic matter present.

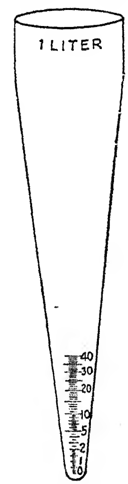


FIG. 127.—Imhoff cone.

By filtering the sewage through filter paper or some other filtering medium before evaporation and ignition, it is possible to determine the quantities of fixed and volatile *dissolved solids* and *suspended solids* either from the filtrate or from the matter deposited on the filtering medium, or from both. This approach does not isolate the *colloidal matter* from the suspended and dissolved solids. Some of the colloidal solids will be reported as suspended solids, but more as dissolved solids. The method of filtration should always be stated. There is at present no simple or standard test for colloidal matter and the term is used somewhat loosely as applying to finely divided matter that does not settle readily.

By *settling solids* is commonly understood the quantity of sludge or sediment, expressed in cubic centimeters per liter, in per cent by volume or in parts per million by weight, which will be deposited from a given quantity of sewage in a certain interval of time. The readiest method of approximately measuring the volume of settling solids is with the aid of an "Imhoff cone," a glass, 4 in. in diameter at the top, 16 in. tall and tapering to the

size of a thimble at the bottom. It is graduated in cubic centimeters at the bottom and holds 1 liter (see Fig. 127). The cubic centimeters of sludge are commonly read after allowing the sample to stand for 2 hours. The method of settling and the time should be noted. If the weight of the settling solids is to be found, which is the only way to make an accurate determination, the supernatant liquid is decanted and its content of total solids determined. The settling solids are then obtained by computing the difference in the total solids content of the unsettled and settled sample.

There is at present no test for floating matters in sewage apart from the determination of oils and fats.

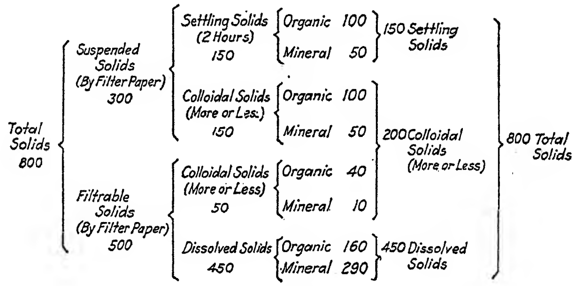


FIG. 128.—Physical condition of principal constituents of sewage of medium strength. (Numbers are parts per million.)

The physical condition of the principal constituents of sewage of medium strength and the average quantities of each are shown in Fig. 128.

254. Organic Matters in Sewage.—The organic matter of animal or vegetable origin in sewage is a combination of the elements carbon, hydrogen and oxygen, together with nitrogen in some cases. Other important elements, such as sulphur and phosphorus, may also be present. The principal groups of organic substances represented in sewage are the proteins, carbohydrates and fats and the products of their decomposition. All of these substances break down more or less readily through the activity of bacteria and other living organisms. Urea, the chief constituent of urine, is decomposed so rapidly into ammonium carbonate that undecomposed urea is only found in quantity in very fresh sewage.

Proteins are the principal constituents of the animal organism. They occur to a lesser extent also in plants. Proteins are

extremely complex in chemical structure and contain as their distinguishing characteristic, besides carbon, hydrogen and oxygen, a fairly high and constant proportion of nitrogen (about 16 per cent), as well as sulphur, phosphorus and iron in many cases. Urea and the proteins are the chief sources of nitrogen in sewage.

The *carbohydrates* include sugars, starches, cellulose and wood fibre. They contain carbon, hydrogen and oxygen in certain definite proportions.

Fats are combinations of fatty acids with glycerin. Soaps are the mineral salts of fatty acids. Fats and soaps are relatively simple in chemical structure and are among the more stable of organic compounds. The fats contain carbon, oxygen and hydrogen and the soaps, as found in sewage, include in addition the more common alkali or alkaline earth metals, such as sodium, potassium, calcium and magnesium.

Winslow and Phelps have estimated the amount of various organic materials in an average American sewage containing 400 p.p.m. of organic matter, one-half being in suspension, the other in solution, to be as follows:

	p.p.m.
Nitrogenous matter	150
Carbohydrates	200
Fats	50

Besides these organic matters of animal or plant origin, there are being discharged into sewers material quantities of so-called mineral oils, products of petroleum and coal tar, that contain essentially carbon and hydrogen.

As shown in Fig. 128 about 67 per cent of the suspended solids and 40 per cent of the dissolved solids are organic in nature. Decomposition of these materials results in a rapid depletion of the oxygen normally present in water, after which foul-smelling compounds are formed. For this reason it is the organic content of sewage, and particularly its avidity for oxygen, that constitutes one of the principal problems of sewage treatment and disposal.

255. The Cycles of Nitrogen, Carbon and Sulphur in Nature.—In order to understand the choice and significance of a number of the tests employed in sewage analysis, especially in relation to the determinations of organic matter, it is necessary to become acquainted with the cycles of nitrogen, carbon and sulphur in Nature. Appreciation of these cycles, too, will be found of

assistance in interpreting the behavior of sewage in treatment works and in streams or other bodies of water into which sewage is discharged.

In its simplest form the *nitrogen cycle* may be idealized diagrammatically as in Fig. 129*a*. As here shown organic nitrogenous matter is decomposed by bacterial activity, and the nitrogen it contained appears first as ammonia. By *oxidation* or *nitrification* this is converted through the medium of two distinct groups of nitrifying bacteria first into nitrites and then into nitrates. Nitrates serve as plant food, and the nitrogen taken up by plants is built into plant tissue, or plant proteins. By death the plant proteins become organic nitrogenous matter once again. If the plants are eaten by animals, part of the nitrogen is converted into animal proteins, part is wasted. The nitrogenous waste products of animal life are organic nitrogenous matter and urea. The latter is broken down into ammonia by another special group of bacteria.

A few deviations from the complete cycle have already been mentioned; there are others. Certain bacteria, for example, reduce nitrates to gaseous nitrogen. This is known as *reduction* or *denitrification*. Other forms of denitrification are the reduction of nitrates to nitrites and ammonia and the liberation of gaseous nitrogen from either of these. By bacterial activity connected with the growth of leguminous plants, furthermore, free nitrogen is utilized in forming plant tissue. This is known as *nitrogen fixation*. Finally, bacteria may convert ammonia into the living matter of their own cells.

It will be noted that the left half of the circle in Fig. 129*a* is associated with living matter, the right with dead or waste matter. The right half is particularly significant in sewage disposal.

The *carbon cycle* is illustrated in Fig. 129*b*. Decomposition of organic carbonaceous matter produces carbon dioxide gas or carbonates. With the aid of chlorophyll, the green coloring matter of plants, and under the stimulus of sunlight, green plants are able to convert carbon dioxide into carbohydrates, which may later be changed to fats and proteins. This use of carbon dioxide is known as *photosynthesis*, the building up of complex substances with the aid of light. As will appear later, plants, while absorbing carbon dioxide, liberate oxygen. In the dark the reverse is true, and oxygen is taken in, while carbon dioxide is given off. This is called *respiration*. When plants die,

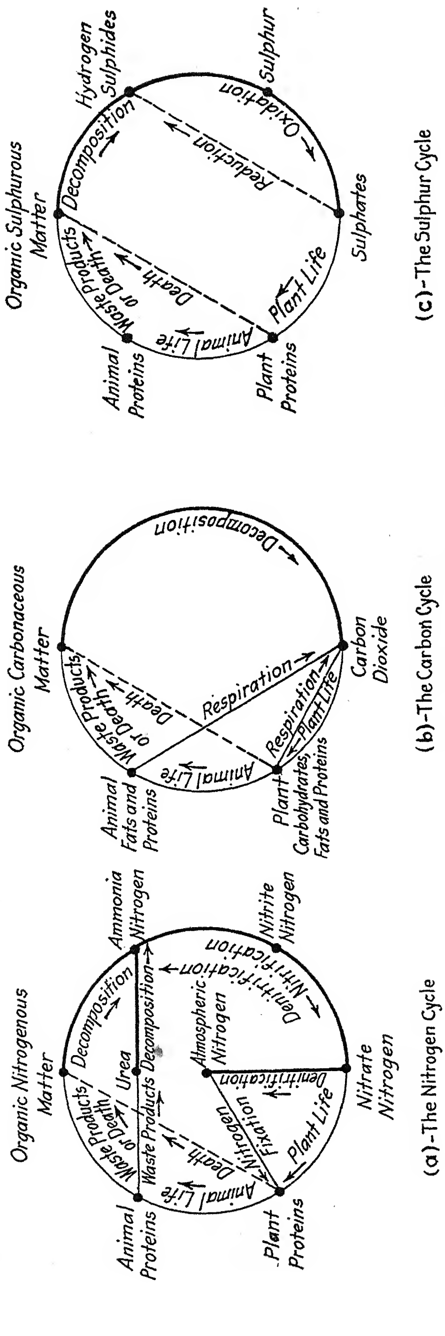


FIG. 129.—The nitrogen, carbon and sulphur cycles in Nature.

the elements of which they are composed recommence the cycle. Animals feeding on plants convert the plant matter into animal tissue and waste products. By respiration, furthermore, animals absorb oxygen while giving off carbon dioxide.

The third cycle that illustrates the rotation in Nature of the elements composing organic matter, is the *sulphur cycle*. Decomposition of sulphur-bearing organic matter in the absence of free oxygen, as shown in Fig. 129*c*, results in the production of sulphides and of hydrogen sulphide, an odoriferous gas. In the presence of oxygen the so-called *sulphur bacteria* oxidize the hydrogen sulphide to produce sulphur and sulphates. The activities of plant and animal life then complete the cycle of this element in Nature. By reduction sulphates can be broken down under conditions analogous to the reduction of nitrates and nitrites, hydrogen sulphide being produced.

Not shown, but associated with practically all the changes just described, is the cycle of water. Bacteria and other primitive forms of life cannot perform their functions in the absence of water. Water, however, does not play a merely passive part by preventing dessication or acting as the conveying medium for food substances; it enters actively into both the synthesis and analysis of organic matter. The addition of water to the molecule is known as *hydrolysis* and commonly results in converting the organic substance, to which water is added, into a simpler, more soluble compound.

It should be noted that in all these changes there is no loss of matter, only a change in its form and characteristics. The fact that all the elements composing organic matter are found at one time or another, alone or in combination, as gases or volatile substances, however, does at times result in their loss from sewage to the atmosphere.

256. Determination of Organic Matters in Sewage.—The tests for organic matters in sewage shown in Table 63 are designed to give information on:

1. The quantity and physical occurrence of organic matter.—Test for volatile solids or loss on ignition (total, suspended, settling, non-settling, dissolved).

2. The quantity and character of nitrogenous matter.—Test for albuminoid and organic nitrogen, which are commonly considered together with the tests for nitrogen in mineral form (ammonia,[1] nitrite and nitrate nitrogen).

[1] Includes both organic and inorganic substances.

3. The quantity and nature of carbonaceous matter.—Test for oxygen consumed.
4. The oxygen avidity of the sewage.—Tests for biochemical oxygen demand and relative stability.
5. The presence of fats and oils.—Test for ether-soluble matter.

The tests for volatile solids or loss on ignition have already been discussed. They do not measure the nature of the organic matter present, but only the state in which organic matters occur.

The nitrogenous matter in sewage is derived from urea and the proteins and their products of decomposition. *Organic nitrogen* is determined by the Kjeldahl method. The sewage is boiled with strong acid which liberates the nitrogen in the complex organic substances. This is converted into ammonia and determined as such. *Albuminoid nitrogen* is supposedly that part of the organic nitrogen which is most readily broken down. It is determined by boiling sewage with a chemical oxidizing agent in an alkaline solution and catching the ammonia distilled off. *Ammonia nitrogen*, formerly called "free ammonia," is driven off when sewage is boiled or distilled without the addition of chemicals. It can also be determined by adding suitable reagents to the cold sample. *Nitrite* and *nitrate* nitrogen are commonly found by adding to the sewage chemical reagents that will impart certain colors to it. These colors are compared with those produced by known quantities of nitrites and nitrates. Other methods are also available such as the chemical reduction of these oxidized products to ammonia and its determination.

There is no reliable test for determination of the carbonaceous matter in sewage. If total carbon could be found readily it would be a valuable measure of total organic matter present. The *oxygen consumed* test, although sometimes thought of as measuring carbonaceous matter, actually gives only the quantity of oxygen that sewage will absorb from potassium permanganate when boiled or heated with acid for a given length of time. It does not give the total oxygen needed for chemical oxidation of all organic matter. The time and temperature employed in this test should be stated.

Decomposition and mineralization of organic matter by bacterial activity are accompanied by depletion of the dissolved oxygen. The avidity of sewage for oxygen, therefore, reflects both the nature and quantity of the organic matter it contains.

There are two tests that measure this characteristic. The first is known as the *biochemical oxygen demand* (B.O.D.) of the sewage; the second as its *relative stability.*

The *biochemical oxygen demand* is commonly ascertained by diluting the sewage with clean stored water whose own B.O.D. has been satisfied, holding the sample at a constant temperature (20°C.) for a certain length of time (5 or 10 days are common periods) and noting the difference in the dissolved oxygen content at the beginning and end of the test. The sample must fill a glass-stoppered bottle, so that no oxygen can be absorbed from the air. This is a delicate test and requires a thorough appreciation of the conditions requisite for its satisfactory conduct.

In the *relative stability* test, use is made of the fact that methylene blue, an organic dye substance, is decolorized when the oxygen in the solution is exhausted. The test is performed by adding a small volume of methylene blue solution to the sewage in a filled glass-stoppered bottle, holding it at a constant temperature (20°C.) and noting the time elapsed before the blue color is discharged. This value is a measure of the *relative stability* of the sample as compared with one which would not discharge its color during the initial stage of oxidation (see section 265).

The reduction in oxygen content in both these tests is due to the activity of living organisms in the sewage. For this reason a disinfected or highly acid or alkaline sewage or effluent must be adjusted to normal conditions and seeded with living organisms before the tests are performed. The laws of oxygen depletion have been formulated and can be used to extend the information gained from either test.

Fats and oils in sewage are derived from animal and vegetable matter or from wastes such as those of garages and gas works. Their quantity is determined as *ether-soluble matter*, by liberating with acid the fatty acids, after evaporating the water, and dissolving them in ether which is then driven off, leaving the ether-soluble matter behind, when it can be weighed.

Aside from the test for fats and oils, the tests for organic matter attempt to measure chiefly the concentration and condition of that portion of the sewage which draws upon the oxygen resources of the bodies of water into which the sewage is discharged. For this reason the tests that measure directly the oxygen avidity of the sewage have come to be the most important. The tests for nitrogen and oxygen consumed are time-honored

tests which have been applied to sewage after being developed for water analysis. Of the nitrogen determinations, the tests for organic nitrogen and for nitrite and nitrate nitrogen are reliable. The tests for albuminoid and ammonia nitrogen are quite unreliable, especially the former. Measuring only a portion of the decomposable substances, the nitrogen determinations yield but scanty information as far as the broader aspects of sewage treatment and disposal are concerned. While they, as well as the oxygen-consumed test, have lost a great deal of their former importance since the introduction of the biochemical oxygen demand test, they are still in general use, largely because of the store of information available through them from the past. Certain of the tests, too, remain of specific value in a number of treatment processes. The test for ammonia nitrogen, for example, appears to be of value under some conditions. Comparison of effluents from treatment works on the basis of nitrification, *i.e.* nitrate production, is not justifiable since it depends upon the quantity of nitrogen originally present in the sewage and, like the other nitrogen determinations, measures only part of the work accomplished. The degree of nitrification is of value, however, in judging the efficiency from some points of view of a particular plant.

257. Mineral Matters in Sewage and Their Determination.— The occurrence and determination of a number of the mineral constituents of sewage, such as the fixed solids and mineral forms of nitrogen, have already been discussed. Most of the remaining forms of mineral matter are only of passing interest.

The *chloride* radicles of substances such as sodium chloride (common table salt) are present in all water supplies, food, kitchen wastes, wash waters, urine and feces. They are readily determined by chemical analysis.

Alkalinity is due to the presence in sewage of the carbonates, bicarbonates or hydroxides of elements such as calcium, magnesium, sodium and potassium, or of ammonium. Of these, calcium and magnesium bicarbonates are most common. Sewage is normally alkaline, receiving its alkalinity from the water supply, the ground water and the sewage matters themselves. Alkalinity is determined by titrating against a standard acid, expressing the result in terms of calcium carbonate.

Sewage which contains mine drainage, wastes from wire-drawing mills, or some other industrial wastes, may be acid; *acidity* is

determined by titrating against a standard alkali, expressing the result in terms of calcium carbonate.

The *hardness* or "soap consuming power" of water is a measure of its content of calcium and magnesium. Since no lather will form until sufficient soap has been added to combine with the salts of these elements, this fact is used for determining hardness. The relation of hardness to total and fixed solids has already been touched upon.

Iron in sewage may originate in the water supply, in the ground water filtering into the sewers, and in industrial wastes, particularly the acid iron wastes of iron works. It is easily determined by chemical analysis. Iron also enters, at times, the underdrains of intermittent sand filters.

Mineral analyses, as far as their wider application to sewage disposal problems are concerned, are of secondary value. The test for chlorides is sometimes used to measure the concentration of sewage. The chloride content undergoes no changes except as diluting water of lower or higher chloride content becomes mixed with the sewage. Knowing the amount of chlorides in the water supply and in the ground water leaching into the sewers, it is possible to judge to some extent the strength of the sewage from this test. The determination, however, is not of sufficient value to be made part of the routine examination of sewage. Of similar local or specific value are most of the other tests for mineral matter, such as alkalinity, acidity, hardness, sulphates, iron and heavy metals. The latter substances are sometimes of importance because of their germicidal action. The tests for alkalinity and acidity in their more general significance have been superseded by the determination of the hydrogen-ion concentration.

258. Gases and Volatile Matters in Sewage and Their Determination.—Odors in sewage and water containing sewage are due to gases or volatile substances produced by decomposition of the organic matter. The most characteristic odor is that of hydrogen sulphide. There are, however, numerous other "sewage" odors, most of which are hard to define. Quantitative measurements of odors cannot be made, though *hydrogen sulphide* can be determined as such. The odor of sewage is examined by shaking a stoppered bottle, half-filled with sewage, and noting the odor while removing the stopper.

Gases are dissolved in water in the same way as are soluble solid substances. They are determined by chemical means

The common gases of the atmosphere, nitrogen, oxygen and carbon dioxide, are found in all waters that are exposed to the air. The amounts taken into solution are limited by the following factors: (1) the coefficient of solubility of the gas in water; (2) the partial pressure of the gas in the atmosphere; (3) the temperature of the water; (4) the purity of the water.

Of these the first three are the most important, as far as fresh water is concerned. The *coefficient of solubility* is the quantity of gas that is absorbed by a unit quantity of water at a given temperature when the water is exposed to a pure atmosphere of the gas under a pressure of 760 mm. The *partial pressure* of a gas is the per cent by volume of the gas in the atmosphere. The quantity dissolved varies directly with the partial pressure. The higher the temperature, however, the smaller is the quantity. The purer the water the greater is its solution power. In sea water with a salinity of 18,000 p.p.m., as measured by the chloride content, the solubility of oxygen, for example, is reduced by about 20 per cent as against fresh water. This is of importance in the disposal of sewage into tidal estuaries.

Example.—Let it be required to find the quantity of oxygen dissolved in water at a temperature of 10°C. exposed to the air, the barometric pressure being 760 mm.

The coefficient of solubility of oxygen as found in tables of chemical constants is 3.84 per cent by volume (one c.c. of oxygen weighs 0.00143 g.). Dry air contains about 20.9 per cent of oxygen by volume. Since equal volumes of gas contain an equal number of molecules this is also the per cent of the pressure, or partial pressure of the oxygen. The vapor pressure of water at 10°C., *i.e.* the partial pressure of water vapor, in the atmosphere in contact with the water surface, is 9.2 mm. Hence the quantity of oxygen that can be dissolved in water at 10°C. exposed to the air is:

$$0.0384 \times 0.00143 \times 0.209 \times \frac{(760 - 9.2)}{760} \times 1{,}000{,}000 = 11.3 \text{ mg. per liter}$$

or p.p.m.

This value could also have been obtained directly from tables such as Table 66. If water or sewage under these conditions contains only 6.0 p.p.m. of oxygen, it is said to be $\frac{6.0}{11.3} \times 100 = 53$ per cent saturated. It should be noted that since the quantity of oxygen necessary to saturate water varies with temperature, this last method of reporting oxygen is confusing unless the temperature is known. If, for example, the oxygen has been used up by biological processes in two sewages whose temperatures are 10 and 20°C. respectively, until the per cent of saturation in both has fallen to 53 per cent, the dissolved oxygen remaining in the first is $0.53 \times 11.3 = 6.0$ p.p.m., while in the second it is only $0.53 \times 9.2 = 4.9$ p.p.m.

When sewage in contact with the atmosphere is only partially saturated with oxygen or with some other gas found in the atmosphere above, it will slowly absorb the gas from the air, the rate of solution depending upon the surface area of the sewage exposed and upon the degree of undersaturation. When sewage contains more of a gas in solution than is consistent with the partial pressure of the gas in the atmosphere the gas will go out of solution until balance is attained. This explains why carbon dioxide, which is commonly present in the atmosphere to the extent of only 0.04 per cent, corresponding to about 1 p.p.m. of CO_2 in the sewage, cannot be used as a measure of oxidized carbonaceous matter in sewage. Too much of it is lost to the atmosphere as soon as it is produced.

TABLE 66.—QUANTITIES OF DISSOLVED OXYGEN IN WATER AT DIFFERENT TEMPERATURES EXPOSED TO AN ATMOSPHERE CONTAINING 20.9 PER CENT OF OXYGEN UNDER A PRESSURE OF 760 MM. INCLUDING PRESSURE OF WATER VAPOR

Temperature, °C.	Parts per million	Temperature, °C.	Parts per million
0	14.6	16	10.0
2	13.8	18	9.5
4	13.1	20	9.2
6	12.5	22	8.8
8	11.9	24	8.5
10	11.3	26	8.2
12	10.8	28	7.9
14	10.4	30	7.6

It is evident from the foregoing that most of the oxygen dissolved in sewage comes from the atmosphere. In water courses containing green plants, however, some of the oxygen may be contributed by photosynthesis. The dissolved oxygen test forms an integral part of the determination of the biochemical oxygen demand of sewage. Used alone it is probably the most significant measure of the pollution and natural purification of streams or other waters. It and the B.O.D. test are probably the most useful pair of determinations that can be employed in sewage disposal problems.

259. The Hydrogen-ion Concentration of Sewage and Sewage Matters.—Water contains, besides molecules of H_2O, free *hydrogen ions* (H^+) and *hydroxyl ions* (OH^-), *i.e.* the water is dis-

sociated. In absolutely pure water the number of H ions equals the number of OH ions, and the water is said to be *neutral.* When the number of H ions exceeds that of the OH ions, the water is said to be *acid.* When the reverse is true, the water is said to be *alkaline.* Since acidity is due to the presence of H ions in a solution, the acidity increases as the number of H ions increases. Conversely, since alkalinity is due to the presence of OH ions in a solution, the alkalinity increases as the number of OH ions increases. A strong acid is one that dissociates highly in water to give a large number of H ions; a strong base (alkali), similarly, is one that dissociates to give a large number of OH ions.

The H ions carry a positive electrical charge, the OH ions a negative one. Hence the concentration of H or OH ions will be reflected in the conductivity of the water and can be measured electrometrically. By the "Mass Law,"

$$\frac{\textit{Concentration of H ions} \times \textit{concentration of OH ions}}{\textit{Concentration of undissociated } H_2O} = \textit{a constant.}$$

In view of the fact that the quantity of undissociated water is extremely large relative to the quantities of H or OH ions, the denominator can be taken as a constant, and we then have

$$\textit{Concentration of H ions} \times \textit{concentration of OH ions} = \textit{a constant.}$$

The constant has been measured and found to be 10^{-14}. Statement of the magnitude of the H-ion concentration, therefore, simultaneously implies that of the OH ions. Hence we commonly refer to the H-ion concentration alone, rather than both the H-ion and OH-ion concentration.

Since pure water contains an equal number of H ions and OH ions, each must have a concentration of 10^{-7}, *e.g.* 1 gram in 10 million liters of water. An acid solution has more H ions, say 10^{-6}, 10^{-5}, etc., while an alkaline solution has fewer H ions, say 10^{-8}, 10^{-9}, etc. For convenience of statement the H-ion concentration is expressed in terms of the exponent which measures the magnitude of the decimal fraction and which is, therefore, the logarithm of the reciprocal of the H-ion concentration. This term, as suggested by Sörensen, is called the "pH value;" hence $pH = \log \frac{1}{H^+}$. It should be remembered that pH 7.0 denotes neutrality, while pH values above 7.0 signify alkalinity and pH values below 7.0 acidity, and that increasing

H-ion concentration, therefore, is associated with decreasing pH values and *vice versa*. In considering pH values it is well to keep in mind, also, that large differences are denoted by small numbers. Thus a difference of 1 in the pH value signifies 10 times the H-ion concentration, and a difference of 2, 100 times the H-ion concentration. Measurements of pH are sufficiently accurate to give values to more than one significant figure; two are commonly employed and three or four in very fine measurements.

In determining pH values in sewage analysis, generally use is made of the fact that certain organic dye-substances, called *indicators*, manifest sensitive color changes at different H-ion concentrations. Comparison of these colors with colors produced in solutions of known H-ion concentration is readily made and will give the desired information. This method of approach is simpler than the electrometric (potentiometric) method.

Formerly the acid or basic properties of water and sewage were determined by finding the amounts of standard acid or alkali necessary to produce color changes in certain indicators, such as methyl orange or phenolphthalein, that are less sensitive than the ones now used. This is known as *titration*, and a solution was spoken of as alkaline to methyl orange or acid to phenolphthalein.

There occur in sewage certain substances known as *buffers*. When acid or alkaline compounds are added to sewage these oppose or offset dissociation, so that in their presence considerable amounts of acid or alkali can be added without changing the pH. This is called *buffer action* and, as will be seen, is an important characteristic of sewage and sewage matters. The determination of H-ion concentration is particularly significant in connection with three problems: (1) the life processes of bacteria that decompose sewage matters; (2) the coagulation and precipitation of suspended and colloidal matter; and (3) the dewatering of sewage sludge. The activities of the micro-organisms of decay are stimulated or retarded by changes in pH. There is an optimum value, and artificial adjustment to this value is sometimes worth while. Coagulation and precipitation of suspended and colloidal solids takes place most rapidly at certain pH values and is therefore aided by bringing the liquid to these values by the addition of acids or alkalis, or by acid or alkaline salts. A similar statement may be made for the dewatering of sewage sludge.

260. The Temperature of Sewage.—The temperature of sewage is commonly higher than that of the water supply, because of the addition of warm water from households and industries. As the specific heat of water is about five times that of the air, the temperatures observed are higher than the local air temperatures during most of the year, and are lower only during the hottest summer months. Since most of our modern sewage treatment works depend upon biological activity, temperature measurements are important. Within the range of temperatures

TABLE 67.—MEAN ANNUAL TEMPERATURES OF AMERICAN SEWAGES

City	Temperature, °F.	Year	City	Temperature, °F.	Year
Chicago, Ill. (Calumet Plant)	53	....	Plainfield, N. J.	60	1925
Gloversville, N. Y.	53	1908–09	Columbus, O.	61	1909–10
Toronto, Ont.	53	....	Philadelphia, Pa. (Northeast Plant)	62	1924–27
Schenectady, N. Y.	55	1927	Pasadena, Cal.	62	
Rochester, N. Y.	57	1927	Cleveland, O. (Westerly Plant)	63	1927
Brockton, Mass.	57	1927	Indianapolis, Ind.	67	1927
Marion, O.	58	1925–27	Durham, N. C.	69	1927–28
Albany, N. Y.	58	1920–21	Decatur, Ill.	80*	
Baltimore, Md.	59	1927			

* Sewers receive unusual proportion of warm industrial wastes.

TABLE 68.—MONTHLY MEAN SEWAGE AND AIR TEMPERATURES, 1927

Month	Schenectady, N. Y.		Cleveland, O.	
	Air, °F.	Sewage, °F.	Air, °F.	Sewage, °F.
January	22	52	27	53
February	24	49	34	53
March	32	48	40	53
April	39	45	47	57
May	48	51	58	62
June	58	54	64	62
July	65	59	71	65
August	73	64	65	72
September	68	64	65	73
October	64	63	56	73
November	55	60	47	64
December	43	54	35	59
Average	49	55	51	63

commonly observed, the higher the temperature the more active are the organisms that bring about the desired changes in the organic matter. The viscosity of sewage decreases with increasing temperature. Temperature measurements will therefore explain the differences in sedimentation efficiencies observed during the various seasons of the year. The variation of oxygen solubilities in water with temperature has already been discussed.

The mean annual temperatures of American sewages are shown in Table 67.

The monthly variation in sewage temperatures for a city without much industrial waste and for one with perhaps an average proportion is shown in Table 68.

261. Living Matters in Sewage.—A great many branches of the animal and plant kingdoms are represented by one or more members in the treatment and disposal of sewage. The most significant part, however, is played by minute living things, too small as individuals to be seen with the naked eye. These, the most lowly organized forms of life, commonly are considered by sanitary engineers as arraying themselves in two groups, the *bacteria*[1] and the *plankton* or *microscopic organisms*. The terms bacterium and plankton are derived from Greek words signifying "a staff" and "wandering" respectively, the first bacteria noted having been staff or rod-shaped organisms and the first microscopic organisms (other than bacteria) having been observed as a free-floating (wandering) assemblage[2] of minute living things. Of the two, the bacteria are the more active, but acquaintance with both groups as well as with the other forms of life encountered in sewage and their functions in sewage treatment and disposal is desirable. These living organisms are of interest particularly from the standpoint of their food habits, which are largely responsible for the changes that are brought about in the so-called *biologically-activated* treatment processes and in the *self-purification of streams*. Leaving a discussion of the bacteria to the next section of this chapter, brief consideration will be given to some of the other organisms encountered.

The *plankton* comprises both plants and animals. The plant-like microscopic organisms fall into two groups, the *algae*[3] and

[1] Singular bacterium.

[2] The term *plankton* denoting an assemblage of organisms is commonly used in the singular, the plural being planktons or, perhaps better, plankta.

[3] Singular *alga*, from the Latin word for "seaweed," a marine alga of large size.

fungi.[1] The former contain chlorophyll in some form or other and, with the aid of sunlight, can elaborate their living matter from the simplest mineral compounds, such as nitrates and carbon dioxide. The latter contain neither chlorophyll nor starch. Lacking these, they are unable to assimilate inorganic matter and consequently live (1) upon dead organic matter or (2) upon a living host. Their mode of nutrition is said to be (1) *saprophytic* or (2) *parasitic.* Strictly speaking, the bacteria are a sub-class of the fungi, which include also the molds and yeasts. Both algae and fungi are found in sewage treatment works, more particularly in trickling filters, where they are active in the decomposition and stabilization of organic matter, as they are also in sewage polluted streams (see Chapter XII).

The animal-like members of the plankton most frequently encountered are classified as *protozoa,*[2] *rotifera*[3] and *crustacea.*[4]

The protozoa are single-celled animals. There are a great many varieties. In sewage, they live upon organic matter and some of them devour bacteria. Their activity is probably of considerable importance, and secondary only to that of the bacteria. As will appear, their mass development seems to be associated in many cases with certain untoward occurrences in sewage treatment processes, such as the "foaming" of Imhoff tanks and the "bulking" of activated sludge. The rotifera and crustacea, like the protozoa, live upon organic debris. The rotifera find much of their food in the algae, and certain crustacea will ingest protozoa. Rotifera and crustacea are more highly organized and appear to be more active in the "natural purification" of streams than in sewage treatment processes.

In sewage treatment plants, a number of different *worms* are active; particularly in trickling filters and activated sludge tanks. *Insects* too occur in trickling filters, both in their developmental and adult stages. Some of them are quite objectionable, others beneficent in their activity. In sewage-polluted streams we have additional varieties of worms and insects, besides *sponges, mollusks, fish* and *higher plants.* All of them play a part directly or indirectly in the satisfactory disposal of waste products.

[1] Singular *fungus,* from the Latin word for "mushroom," a fungus of large size.

[2] Singular *protozoon* from the Greek words signifying "first, most primitive animal."

[3] Singular *rotiferum* or rotifer, plural also rotifers, from the Latin signifying "carrying a wheel."

[4] Singular *crustaceum,* derived from the Latin word for "shell."

Gulls and other birds which feed upon floating organic matter discharged from sewers, and rats, the scavengers of sewers, in which many live, are worthy of only passing mention.

Identification of planktonic and higher forms of life is a task for which few engineers are equipped by training. Quantitative methods, excepting for the plankton, are commonly confined to enumeration of the organisms found. Plankton at present are reported in *cubic standard units.* This unit is defined as equivalent to 8,000 cubic microns. To obtain parts per million by volume, the number of cubic standard units must, therefore, be divided by 125.

262. The Bacteria.—Bacteria are minute single-celled, plant-like organisms, seldom more than 2μ or 3μ in their greatest dimensions. In form, they occur as balls, bats or corkscrews, called *cocci, bacilli* and *spirilla.*[1] They are found singly or in groups. Some of the cocci arrange themselves in pairs, in packets or in bunches (like grapes). Other cocci form chains, as do some of the bacilli. The cocci are non-motile; the bacilli and spirilla are either motile or non-motile. Fine hair-like or whip-like appendages[2] on the entire body surface or at the ends of the cell constitute the organs of locomotion.

Bacteria multiply by *fission*, a division of the cell into two practically equal portions. Under favorable conditions, they increase with great rapidity, which accounts in part for the large amount of work accomplished by them.

The growth and activities of bacteria are dependent upon the consumption of food, as is the case with every other form of life. According to their food habits, bacteria are classified as saprophytes and parasites. The former are of greatest importance in sewage treatment and disposal; the latter are significant from the hygienic standpoint. The organic and mineral matter in sewage usually affords suitable food for the development of saprophytic bacteria. Some parasitic organisms can also live on dead organic matter. Most parasitic organisms, however, do not develop to any extent outside the body of their host. The disease-producing (pathogenic) bacteria die off quite rapidly in sewage. The bacterial cell is composed of an enclosing wall and the living matter (protoplasm) inside. The protoplasm can extract and assimilate nourishment from the food reaching it to obtain vital

[1] Singular coccus, bacillus and spirillum, respectively.

[2] These are called *cilia*, singular *cilium*, and *flagella*, singular *flagellum*, respectively.

energy, or to form new cell wall and more protoplasm which result in growth. The cell wall permits transfusion of soluble substances, but keeps out solid particles. Since much bacterial food is in solid form, the organisms must dissolve this food outside the cell. This they do by substances called *ferments* or *enzymes*, carried and secreted by the cell. Enzymes are catalysts, *i.e.* substances which promote chemical reactions without themselves entering into the reactions.

During disintegration of food substances by bacteria, acids and other products injurious to them are commonly formed. These may accumulate until further multiplication is stopped. Checks to bacterial growth are also offered by an insufficient food supply, unsuitable temperature, competition of different species of bacteria, ingestion by other organisms such as the bacteriverous protozoa, and the presence of certain poisonous chemicals, such as chlorine compounds, acid or caustic substances, copper salts and arsenic. In an unfavorable environment some bacteria, assuming that the cells are not actually destroyed, have the ability to form spores, a resting, more resistant stage, analogous to the hibernation or estivation encountered among higher forms of life. During growth, many of the bacteria associate in sticky, jelly-like masses of organic matter called *zoöglœa.*[1] Other organisms may also be included.

Bacteria, like all other living organisms, need oxygen to carry on their life processes. Life, indeed, is a process of combustion. Some bacterial organisms require free oxygen and obtain it from the oxygen dissolved in the water. These are called *aerobic*[2] bacteria. Others can obtain their oxygen supply from the oxygen radicles of organic compounds or such mineral substances as nitrites, nitrates and sulphates. These are called *anaerobic* bacteria. Many bacteria are neither strict aerobes nor strict anaerobes and are said to be *facultatively* aerobic or anaerobic. This relation to oxygen is shared by other primitive forms of life such as the protozoa and rotifera. The higher the scale of life, the more strictly does it become aerobic. Classification of bacteria with respect to oxygen conditions is particularly important in sewage problems. The tremendous development of bacteria and other organisms in sewage often depletes the dissolved oxygen and establishes so-called *anaerobic* or *septic conditions.*

[1] From the Greek words for "animal" and "glue."

[2] From the Latin and Greek words for "air" and "life."

An important characteristic of bacterial growth is that the cells are sensitive to the *reaction* of their environment, *i.e.* its acidity or alkalinity as measured by the H-ion concentration. A favorable reaction will promote growth, an unfavorable one will arrest it or even destroy the organisms completely. Bacteria are likewise sensitive to the temperature of their environment. It has been stated that the growth is checked by unsuitable temperatures. They are destroyed at high temperatures and their activities are inhibited at low ones. Each species has its optimum temperature, or temperature of best growth. For some it is relatively high; for others relatively low.

263. Determination of Bacteria in Sewage.—Bacteria find their way into sewage through many channels. They are present in the water supply of the community as well as in the ground water leaching into the sewers. Some doubtless fall into the sewage from the air. The wastes of certain industries contain large numbers. Many more are washed into sewers by storm water. By far the greatest number, however, come from human and animal excreta, which teem with these minute living cells.

Determination of bacteria in sewage, as in water, makes use of the fact that bacteria can be cultivated on suitable nutrient media and that different groups of bacteria will react differently to artificial environmental conditions. Three tests are in common use:

1. The count of bacteria that develop on agar or gelatin at 20°C. in 48 hours.
2. The count of bacteria that develop on agar at 37°C. in 24 hours.
3. The quantitative test for *B. coli.*

Agar and *gelatin* are so-called solid culture media. They contain besides certain nutrient substances (meat extract and peptone) enough agar-agar, a Japanese seaweed with gelatinous properties, or gelatin, itself a nutrient substance, to render the mixture solid at the temperatures of incubation. If a known volume of sewage is placed in a sterile glass dish and melted agar or gelatin is added, a solid medium, in which the sewage bacteria are entrapped, results upon cooling. The organisms are isolated thus at the same time that they are provided with an abundant food supply. If the glass dish is covered to keep out bacteria that might fall into it from the air and is held (incubated) at a

favorable temperature, single bacteria that were invisible to the naked eye will multiply and form colonies so large that they can be seen easily without the aid of a microscope. A count of the colonies, therefore, will show the number of bacteria originally present in the sewage that will develop under the conditions of test. No method is known for readily ascertaining the exact number of bacteria in a liquid. The 20° and 37° counts merely give the numbers of organisms that develop on the particular types of culture medium at the specified temperatures in the particular time. Many of the most important sewage bacteria cannot grow under these conditions and are, therefore, not enumerated. The 20°, or room-temperature, count is supposed to measure the normal saprophytic *flora* of sewage. The 37° count is employed to enumerate those organisms that grow at the body temperature of man and warm-blooded animals and represents, therefore, in a general way the organisms of parasitic origin. As a matter of fact, no rigid demarkation of bacterial groups can be obtained by these two tests.

The quantitative test for *B. coli*,[1] if carried to completion, is probably the best index of sewage pollution that we now possess. *B. coli* has been chosen the standard indicator organism because it occurs in large numbers in the excreta of man and the higher animals and is readily identified. While parasitic, it is not pathogenic, though closely related to the causative organism of typhoid fever. The test for *B. coli* is based on the fact that this organism ferments milk sugar (lactose) to produce hydrogen and carbon dioxide gas, at the same time rendering the medium acid. Gas-production in a nutrient medium containing lactose is therefore presumptive[2] evidence of the presence of *B. coli*. By adding definite volumes of water or sewage to lactose broth quantitative information can be obtained, such as that coli is (presumably) present in 10 cc. but not in 1 cc.

Outside of these simple tests, the isolation, identification and enumeration of the various species of sewage bacteria are usually undertaken only in connection with research problems. It is possible, however, to draw conclusions from the changes wrought in sewage by treatment processes, and by the natural purification agencies of streams as to the presence and activity of the most important groups of organisms.

[1] An abbreviation of Bacillus coli, recently renamed Escherichia coli after its discoverer, Escherich.

[2] For the complete test see Standard Methods for the Examination of Water and Sewage.

Bacterial counts are expressed in numbers per cubic centimeter. *B. coli* results are reported per 100 cc.

The determination of the number of bacteria and the test for *B. coli* are used chiefly in connection with problems of water contamination. Here the information obtained is more directly significant than it is in sewage treatment.

The number of bacteria in sewage varies with the season of the year, the age of the sewage at the place the sample was taken, its temperature and dilution by water. Counts of several million bacteria per cc. are common. The number of bacteria is also affected by industrial wastes discharged into sewers. Hoskins[1] gives the following average values for several American cities:

TABLE 69.—BACTERIA ADDED TO STREAMS BY THE SEWERED POPULATIONS OF CINCINNATI, LOUISVILLE, CHICAGO AND PEORIA

Season	Billions of bacteria per capita per day		
	Gelatin at 20°C. for 48 hours	Agar at 37°C. for 24 hours	B. coli
Summer	17,067	21,362	383
Winter	4,083	2,600	124
Year	12,240	12,676	249

THE DECOMPOSITION OF SEWAGE

264. Aerobic and Anaerobic Decomposition.—Much information remains to be obtained before exactly what takes place during the decomposition of sewage can be stated. In general, two types of decomposition are distinguished, *anaerobic decomposition* or *putrefaction* and *aerobic decomposition* or *oxidation*. The first is brought about mainly by organisms which thrive in the relative absence of free oxygen, the second by organisms which require free oxygen to promote their activities. Among the organisms, bacteria predominate.

Putrefaction is characterized by end products such as methane (CH_4), hydrogen (H_2), hydrogen sulphide (H_2S) and carbon dioxide (CO_2), and by intermediate compounds which often are volatile and offensive in nature. The end products of oxidation

[1] U. S. Pub. Health Rept., 1926; **41**, 321.

are carbon dioxide (CO_2), nitrates (NO_3), sulphates (SO_4) and other inoffensive substances. The decomposition of the nitrogenous, carbonaceous and sulphur-bearing compounds in sewage has been discussed in connection with the rotation of these elements in Nature, and the part played by bacteria was pointed out. The activity of these organisms is interlocking, the waste products of the nutrition of one group of organisms being used by another. The organisms work, so to speak, in relays. There is co-operation, but also much antagonism.

The decomposition of sewage is generally the result of both putrefactive and oxidizing processes. Putrefaction is the first stage, oxidation the second. Putrefaction does not necessarily imply total absence of free oxygen. It may proceed in the presence of free oxygen inside the masses of sewage matter without, however, giving rise to objectionable conditions. Strictly anaerobic conditions are only found when long storage of sewage or sewage matters in so-called *septic tanks* or *sludge-digestion tanks* depletes the dissolved-oxygen content of the sewage, or when streams or other bodies of water are *overloaded* with sewage. Septic conditions are often produced, however, in long outfall sewers and parts of the sewerage system in which steady or self-cleaning flow is not maintained. It is important to remember that as long as any oxygen remains in solution, decomposition, even though putrefaction is in process, will not give rise ordinarily to offensive conditions.

The decomposition of sewage does not result in complete mineralization of all of the organic matters present. A large proportion, particularly of the sewage solids, is converted into humus-like organic compounds that are relatively stable in character. Both types of decomposition are made use of in sewage treatment, putrefaction being employed chiefly for the destruction of the settled solids or sludge, oxidation for the colloidal and dissolved matter. The course which decomposition runs under strictly anaerobic conditions will be discussed further in Chapter XIV in connection with sludge digestion. Decomposition in the presence of free oxygen, which is of general interest in the interpretation of sewage analyses as well as of sewage treatment and disposal processes, will be taken up in the next section of this chapter, as well as in Chapter XII.

265. The Oxygen Requirements of Decomposing Sewage.— The oxygen requirements of fresh decomposing sewage, as

measured by the biochemical oxygen demand (B.O.D.) test, are illustrated in Fig. 130. As here shown there are two stages. At 20°C., for example, the first stage extends to about the sixteenth day and is characterized by a gradual falling off in the quantity of oxygen used up in equal time intervals. A transition is then made to the second stage, during which transition the rate of oxygen consumption increases until about the twentieth day, after which the rate, although lower than during the transition stage, continues to be fairly high and constant for a protracted period of time. The second stage is known as the *nitrification* stage, because, during it, oxidation of the nitrogenous organic matter to nitrates takes place. At higher temperatures, the first stage is shortened in time; at lower ones, it becomes longer.

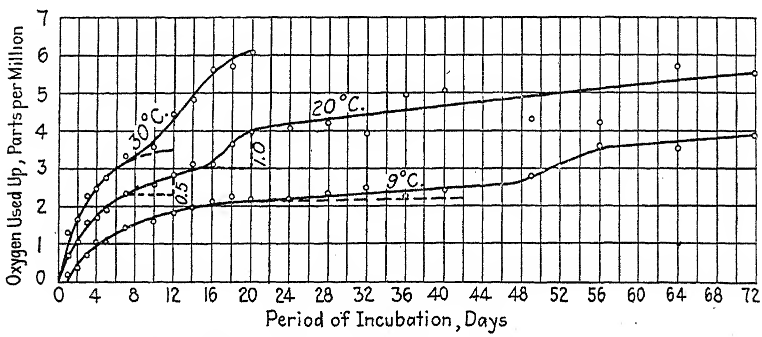


FIG. 130.—Oxygen requirements of fresh sewage.

Mathematical studies of the deoxygenation curve show that during the first stage the consumption of oxygen is proportional to the oxygen requirement of the sample at any time, *i.e.* to the amount of the undecomposed organic matter remaining.

This can be formulated as follows:

If L = B.O.D. in p.p.m. exerted by organic matter during first stage

L_t = B.O.D. in p.p.m. remaining after time t

$L - L_t = X_t$ = Oxygen in p.p.m. used up in t days, as determined by the B.O.D. test

$$-\frac{\dfrac{dL_t}{L_t}}{dt} = K', \text{ where } K' \text{ is a constant}$$

$$-\int_{L}^{L_t}\frac{dL_t}{L_t} = K'\int_0^t dt$$

$$\log \frac{L_t}{L} = \log \frac{L - X_t}{L} = -K_1 t,$$

where $K_1 = 0.4343\ K' =$ the deoxygenation constant

$$\frac{L_t}{L} = \frac{L - X_t}{L} = 10^{-K_1 t}$$

$$X_t = L - L_t = L(1 - 10^{-K_1 t})$$

Chemists call this a *unimolecular reaction* because the velocity of the reaction is controlled only by the concentration of the oxygen-requiring substances, the amount of oxygen present being sufficient to oxidize these substances as soon as they are freed. The formula developed by calculus is merely a variant of the expression for compound interest

$$L_t = L\ (1 + r)^t,$$

where
L_t = capital after time t
L = original capital
r = rate of interest

The B.O.D. decreases at a uniform rate, the decrease being determined in a unit of time by the B.O.D. remaining to be satisfied after each unit of time, just as money increases at a uniform rate, the increase in a unit of time being determined by the capital accumulated to draw interest after each unit of time. Since a decrease and not an increase takes place,

$$L_t = \frac{L}{(1 + r)^t} = L(1 + r)^{-t}$$

$$\log \frac{L_t}{L} = -t \log\ (1 + r) = -K_1 t$$

The deoxygenation constant K_1, it has been determined, is 0.100 at 20°C. The rate of deoxygenation naturally changes with temperature as do all biologically-activated processes. From considerations of physical chemistry and actual test the deoxygenation constant K_1 at any temperature T, $K_{1(T)} = K_{1(20)}[1.047^{T-20}]$. Temperature, furthermore, affects the magnitude of the first stage B.O.D., and the first-stage demand L at any temperature T has been found to bear the following relation to the first-stage demand at 20°C.: $L_T = L_{20}[1 + 0.02(T-20)]$.

Example.—An example will illustrate the use of these equations. Let it be required to find the 1-day, 37°C., B.O.D. of a sewage whose 5-day, 20°C., B.O.D. is 100 p.p.m. The first-stage 20°C., B.O.D. is found as follows:

Since

$$\log \frac{L - X_t}{L} = -K_1 t,$$

$$\log \frac{L - 100}{L} = -0.100 \times 5 = 0.500 - 1$$

$$\frac{L - 100}{L} = 0.316$$

$$L = \frac{100}{0.684} = 148 \text{ p.p.m.}^{[1]}$$

The first-stage 37°C. B.O.D. is therefore:

$$L_{37} = L_{20}[1 + 0.02(T - 20)] = 148[1 + 0.02(37 - 20)] = 198 \text{ p.p.m.}$$

and since

$$K_{1(37)} = K_{1(20)}(1.047^{T-20}) = 0.100(1.047^{17}) = 0.218,$$

$$\log \frac{198 - X_t}{198} = -0.218 \times 1 = 0.782 - 1$$

$$\frac{198 - X_t}{198} = 0.605 \text{ and the 1-day, 37°C., B.O.D., } X_t = 78 \text{ p.p.m.}$$

If the oxygen present in a sample of sewage is used up in t days by biochemical activity, the B.O.D.[2] of the sample (X_t) must be equal to the oxygen originally available. The percentage ratio of the oxygen available (X_t) to the oxygen required during the first stage (L) is then called the relative stability (S) of the sewage. At 20°C., $S = \frac{X_t}{L}100 = 100(1 - 10^{-0.1t}) = 100(1 - 0.794^t)$.

Table 70.—Relative Stability Numbers

(Standard Methods for the Examination of Water and Sewage. American Public Health Association, 1925)

Time required for decolorization at 20°C., days	Relative stability, per cent	Time required for decolorization at 20°C., days	Relative stability, per cent
0.5	11	8.0	84
1.0	21	9.0	87
1.5	30	10.0	90
2.0	37	11.0	92
2.5	44	12.0	94
3.0	50	13.0	95
4.0	60	14.0	96
5.0	68	16.0	97
6.0	75	18.0	98
7.0	80	20.0	99

[1] This result could also have been found from Table 70, $L = \frac{100}{0.68} = 148$ p.p.m.

In parts per million.

This relationship is the basis of what is called *relative stability* and is employed chiefly in connection with the methylene blue test (see Section 256). If $t = 10$, for example, $S = 100(1-10^{-1}) = 90$, and the relative stability of a sample of sewage that does not decolorize methylene blue at 20°C. until 10 days have elapsed is said to be 90 per cent. The relative stability numbers obtained in this way for a temperature of 20°C. are shown in Table 70.

It is evident from the shape of the deoxygenation curve that the equations just discussed and the relative stability table can be used in computing the B.O.D. for a given period from the 5-day or 10-day B.O.D. tests only when decomposition of the sewages examined lies within the period of the first stage. Use of the relative stability numbers in Table 70, it must also be remembered, is restricted to 20°C. tests.

CHARACTERISTICS AND BEHAVIOR OF SEWAGE IN RELATION TO ITS TREATMENT AND DISPOSAL

266. Outline of Sewage Treatment and Disposal Methods.—The following outline will serve to show the relation of the common sewage treatment and disposal methods to the character and behavior of the various constituents of sewage that have been discussed in this chapter. The outline is merely suggestive and exceptions to some of the statements should be noted by the student.

Sewage Disposal Methods[1]

A. Disposal without treatment.
- 1. Dilution or disposal into water (including fish ponds).
- 2. Irrigation or disposal on land (including subsurface irrigation).

B. Disposal after treatment by one of the following methods or a combination of these methods.
- 1. Separation of solids and liquids.
  - (*a*) Floating solids and coarse suspended solids by
    - (1) Screens.
    - (2) Skimming tanks.
  - (*b*) Grit by grit chambers or detritus tanks.
  - (*c*) Coarse and fine suspended solids by
    - (1) Sedimentation.
      - (*a*) Plain sedimentation (settling solids).
      - (*b*) With chemical precipitation (suspended and colloidal solids).
    - (2) Filtration and activation (see "Treatment of liquid").

[1] Adapted from *Am. Jour. Pub. Health*, **15**, 334.

2. Treatment of liquid by
   (*a*) Oxidation through
      (1) Dilution.
      (2) Irrigation.
      (3) Sand filtration.
      (4) Contact beds.
      (5) Trickling filters.
      (6) Activation.
   (*b*) Disinfection.
3. Disposal of the effluent by
   (*a*) Dilution.
   (*b*) Irrigation.
4. Treatment of solids by
   (*a*) Digestion.
   (*b*) Dewatering with or without digestion.
5. Disposal of solids.

267. Significance of Analyses in Sewage Treatment and Disposal.—Broadly speaking, it is the aim of sewage treatment (1) to remove the suspended or readily settling portion of the sewage matters as an inoffensive or even marketable sludge and (2) to oxidize the soluble and colloidal portion. It should be the aim of sewage analyses, therefore, to provide information upon which treatment methods can be based and from which the efficiency of performance can be gaged.

1. Removal of the suspended portion is accomplished by *screening* or by *sedimentation*, the former collecting only the coarser particles, the latter those matters that will readily settle when the velocity of flow is checked. The *fresh solids* obtained, being in part organic in nature, remain offensive. Before they can be disposed of, therefore, it is commonly necessary that they be rendered stable. This is accomplished best by permitting them to decompose at the plant in what are called *digestion tanks*. Design of screens and of sedimentation and digestion tanks, therefore, requires a knowledge of the quantities and nature of the suspended matters in sewage. It is necessary to know how much material probably will be removed by screening or sedimentation; what the screenings or sludge will be like; how much material must be handled, dried and finally disposed of.

2. Oxidation of the soluble or colloidal portions is accomplished by *filtration* or *aeration*, which may remove and at the same time dispose of suspended or colloidal matters not removed by sedimentation. Analyses of the sewage must therefore furnish information upon the load to be handled by these devices.

This is gaged by the analyses for organic matter in the sewage and its oxygen demand. In the past, determinations of nitrogen and of carbonaceous matter have been used extensively for this purpose. In recent times the amount and condition of the organic matter are being measured more directly by the *biochemical oxygen demand* test.

Destruction of the organic matter in sewage treatment is brought about by the activities of hosts of living organisms, particularly bacteria. Although they are, therefore, of greatest importance, the identification and enumeration of these organisms is seldom attempted except in a rough way because of the difficulties involved. Sewage treatment processes, however, are selected and controlled with a view to providing the desirable forms of life with environmental conditions in which their activities will be promoted. Among the analytical determinations that are of assistance are those of temperature and hydrogen-ion concentration.

Performance of the various treatment processes is gaged by operating records that give the analyses of the untreated and treated sewage. The efficiency of each process is determined by suitable tests whose judicious selection is, therefore, an important matter.

Sewage, whether treated or not, must ultimately be disposed of either on land or into water. The former is known as *irrigation*, the latter as *dilution*. Most commonly, sewage is discharged into streams, lakes, or tidal estuaries. Here the sewage undergoes the natural purification processes, active in all waters, standing or flowing. The processes of self-purification are much like those that have been developed for the artificial cleaning of sewage in treatment works. The load of sewage which a given body of water can receive without the creation of nuisances is strictly limited, as will be shown in the succeeding chapter.

In sewage disposal the amount and character of the suspended solids and the oxygen requirements are again the most significant information to be obtained. Knowing the character of the water course into which the sewage is discharged and the uses to which its waters are put, these tests may be employed to determine whether the sewage can be discharged in a raw state or whether it must first be treated, and how far the treatment processes must be carried. It should be remembered that the real object of sewage treatment is to prepare the sewage for disposal. The

fact that many water courses are the sources of drinking water supplies or are used for recreational purposes introduces the use of most of the common tests germane to a water analysis. These, however, are performed on the water rather than on the sewage it receives.

In the operation of treatment and disposal works the information presented in sewage analyses will serve a fourfold purpose as follows:

1. To measure the efficiency of plant operation and of the various treatment processes employed.
2. To indicate modifications of operation methods for greater efficiency.
3. To provide information for the design of extensions to the plant or for the design of other similar plants.
4. To establish the character of the effluent from the plant, particularly in regard to stream pollution and the rights of riparian owners on the waters receiving the treated sewage.

268. Sewage and Disease.—Sewage may contain the organisms of any of the infectious diseases of man in which the causative organism passes out of his body into his water-carried wastes. Most important of these diseases are the so-called gastro-intestinal infections, notably typhoid fever, dysentery, the diarrheas, and in oriental countries, cholera. Persons suffering from these diseases and so-called chronic carriers, persons who have recovered from the disease but continue to carry the germs in their body (sometimes for life), discharge enormous numbers of these pathogenic organisms through their stools or urine. Sewage pollution of water supplies, bathing beaches, and shellfish beds, therefore, may result in epidemics of typhoid fever or other intestinal diseases. From the hygienic standpoint, the fate of pathogenic organisms in sewage is the most important problem in sewage disposal. It is assumed therefore, for sake of safety, that germs of disease are always present in sewage.

Fortunately pathogenic bacteria decrease rapidly in sewage and sewage-polluted water. Accustomed to the abundant food supply and favorable environment of the human body, they cannot survive for a long period of time in sewage. The forces active in their destruction are many. Being attached to suspended matter, some are removed from the sewage by sedimentation. Some are ingested by predatory protozoa and other animals and, since the fauna of sewage is more abundant than that of clean water, they are the more quickly destroyed. Lack of

suitable food causes a gradual disintegration of the cell and, as cell activity is greater at higher temperatures, survival is of less duration in warm sewage than in cold water. Cleaner water, however, permits farther penetration of the sun's rays, the ultraviolet portion of the sun's spectrum constituting a potent disinfectant of polluted waters. Due to factors such as these, pathogenic organisms die away, the various forces combining to reduce their numbers gradually so that in time they are all destroyed. The rate of death under a given set of conditions is fairly constant, and the number destroyed in a unit of time is, therefore, proportional to the number surviving. The longer the time, the fewer survive.[1]

269. Industrial Wastes.—In sewered communities, industrial wastes are often discharged into the public sewers and impose an additional burden on sewage treatment and disposal works which must be recognized in their design. The characteristics and behavior of many of these wastes are similar to those of domestic sewage, but there are others that may interfere seriously with the operation of sewage treatment plants and outfall works. Among the industries producing large quantities of wastes are food manufactories, such as stock yards, packing houses, corn products plants, sugar refineries and breweries, and plants for tanning, wool-scouring, wool washing, cloth washing, dyeing and bleaching, wire drawing and galvanizing, pulp- and paper-making

TABLE 71.—QUANTITIES OF INDUSTRIAL WASTES IN A FEW AMERICAN CITIES[2]

City	Sources of wastes	Gal. per capita daily	Per cent of total sewage
Fitchburg, Mass.......	Paper Mills	262	68[3]
	Woolen Mills	22	6[3]
	Gingham Mills	5	1[3]
	Total.................	289	75[3]
Milwaukee, Wis.......	Breweries, Canneries and Packing Houses	....	35
Gloversville, N. Y. ...	Tanneries	47.5	38
Cincinnati, O:.... ...	Breweries, tanneries, packing houses, soap factories	47.5	31

[1] Formulation of the way in which pathogenic bacteria die away in sewage follows much the same lines as those given in connection with the deoxygenation of sewage.

[2] Figures represent conditions existing at time of investigations, *not* present conditions.

[3] These industrial wastes were not discharged into the sewers; the figures represent the ratios, in percentage, between the quantities of waste and of sewage inclusive of the industrial wastes.

and gas-making. The effect of industrial wastes on the composition of municipal sewage can be gaged from the data presented in Table 64.

The quantity of wastes produced depends upon the character of the industries, and the size of the plants. The estimated quantity of industrial wastes in certain cities is given in Table 71; not all of them are now discharged into the sewers, and there have been changes in the industries since the estimates were made.

More important than the increase in volume of sewage produced by industrial wastes is the amount and character of the waste materials they contain. The authors have made the following rough estimates for a number of American cities (Table 72).

TABLE 72.—PERCENTAGE INCREASE IN SUSPENDED SOLIDS AND OXYGEN CONSUMED DUE TO INDUSTRIAL WASTES

City	Kind of waste	Increase in per cent	
		Suspended solids	Oxygen consumed
Akron	Rubber-reclaiming wastes	120	140
Chicago (packing house district only)	Packing house wastes	460	225
Dayton	Paper mill wastes	60	25
Fort Worth	Packing house wastes	65	
Gloversville	Tannery wastes	155	115
Milwaukee (portion of city)	Packing house and tannery wastes	70	10

The nature of industrial wastes in relation to their disposal is made particularly evident by studies of their biochemical oxygen demand values. At Chicago, for example, it was estimated that in 1920 the oxygen demand of industrial wastes was equivalent to that of a population of 1,500,000 persons, the human population of Chicago being 3,000,000.

Certain industrial wastes interfere with sewage treatment and disposal by reason of their bactericidal properties. Thus, at New Haven, Conn., it was found that the copper wastes from wire works and other copper-working establishments rendered

the use of biologically-activated sewage treatment processes unsatisfactory. At Chicago (Calumet plant) the wastes from paint industries containing copper, arsenic and lead compounds interfered seriously with the activated-sludge treatment of the sewage until controlled. Sulphur wastes from iron works and paper mills may lead to the production of odors of hydrogen sulphide; acid wastes may affect sludge digestion adversely; oily wastes may interfere with the activated-sludge processes and gas-plant wastes may give rise to a variety of troubles.

The question of industrial wastes will be considered further in Chapter XX.

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Problems

1. If a sample of sewage from a city of 50,000 people with a sewage flow of 80 gal. per capita daily contains 171 p.p.m. of settling solids (2 hours) and the loss on ignition of these solids is 41 p.p.m., (*a*) how many tons of settling solids will be produced per 1,000 population per year and how many of these will be organic in nature; (*b*) how many pounds of mineral and of organic solids will be deposited by 2 hours' settling from a million gallons of sewage? *Ans.* (*a*) 21 tons, of which 5 tons will be organic. (*b*) 1,080 lb. of mineral and 340 lb. of organic solids.

2. Assuming that the average quantity of sewage of a community is 100 gal. per capita daily, prepare a diagram similar to Fig. 128 showing the physical condition of the principal constituents of sewage of medium strength expressed in grams per capita per day.

3. The solubility of oxygen in water at 20°C. is 3.14 per cent by volume, and the vapor pressure of water is 17.5 mm. at this temperature. (*a*) How many p.p.m. of oxygen will pure water exposed to the average outdoor air contain at this temperature when the barometer reads 760 mm.? (*b*) What will be the per cent of saturation of the dissolved oxygen if the water contains 4.6 p.p.m.? *Ans.* (*a*) 9.2 p.p.m. (*b*) 50 per cent.

4. The solubility of carbon dioxide in water, as given by Bohr and Bock, is 0.232 per cent by weight at 10°C. when the partial pressure of the gas plus the vapor pressure of water at the temperature indicated is 760 mm. How much CO_2 will be found in the water if the air in contact with it contains 10 parts per 10,000 of CO_2 by volume when the barometer reads 750 mm.? *Ans.* 2.3 p.p.m.

5. The 5-day 20°C. biochemical oxygen demand of fresh sewage is 100 p.p.m.

(*a*) What will be its demand during the first stage?

(*b*) What will be the 10-day 20° demand?

(*c*) What was its 1-day 20° demand?

(*d*) Had the bottle been incubated at 30°C., what would the 5-day 30° demand have been? *Ans.* (*a*) 146 p.p.m. (*b*) 132 p.p.m. (*c*) 31 p.p.m. (*d*) 148 p.p.m.

6. How many pounds of oxygen are required to prevent anaerobic conditions during 3 days of flow, in a stream receiving settled sewage whose 5-day, 20°C., B.O.D. is 20 p.p.m., assuming that 1 m.g.d. of sewage is discharged and that the temperature of the water is 25°C.? *Ans.* 157 lb.

7. The 5-day, 20°C., oxygen demand of an industrial waste was found to be 400 p.p.m. The volume of the wastes is 200,000 gal. per day. If the daily per capita B.O.D. is assumed to be 50 grams on the basis of the 5-day test, what is the population equivalent of the industrial wastes? *Ans.* 6,060.

CHAPTER XII

SEWAGE DISPOSAL BY DILUTION

270. Definitions.—There are but two possible methods of final disposal for the sewage collected from cities and towns, namely into water and upon land. The first is called *dilution;* the second, *irrigation.* Historically, the disposal of sewage by dilution is an outcome of the development of sewerage systems from the drains which have been used since the inception of city life for carrying away storm and ground water from street, yard and cellar areas. Naturally these drains were laid so as to drain into the nearest water course, and when the practice was adopted, about a century ago, of discharging into them human excreta and other household wastes, sewage disposal by dilution had its inception. To-day it remains by far the most common method of disposing of municipal sewage and water-carried industrial wastes.

Irrigation is used only to a small extent, and then very largely as a treatment process producing an effluent or seepage into water.

Dilution is employed both for the disposal of raw sewage and of treated sewage. In 1915 the authors estimated that 42.3 per cent of the 99 million persons in the United States were served by sewerage systems, 26.7 per cent discharging their sewage untreated into lakes and streams, 8.6 per cent disposing of it into the sea, and 7.0 per cent treating it before disposal by dilution. Records of the sewerage works of the cities with populations in excess of 100,000 in 1920, show that in 1926 the water-carried wastes from 82 per cent of the population of these cities was discharged into water without treatment other than fine screening in a few instances. The magnitude of the problem in its various phases is therefore evident.

Sewage disposal by dilution is not, as its name seems to imply, merely a means for getting rid of sewage matters by diluting them with or dispersing them in large volumes of water so that their origin can no longer be recognized. That is only one aspect of the process, whose chief merits are associated with the fact that there are operative in sewage-polluted waters the same forces of purifi-

cation which are employed artificially and more intensely in sewage treatment plants. We might better speak of sewage disposal by dilution as "treatment by natural purification in water."

271. Limitations of the Process.—Streams, lakes or tidal estuaries into which sewage is discharged are limited in the *pollutional load* they are able to receive without giving rise to objectionable conditions, just as artificial treatment processes are limited in the amount of waste matter they can effectively handle. The limiting loads depend upon the physical, chemical and biological conditions of the water into which the sewage is discharged, conditions whose relative importance is being determined by studies of the so-called self-purification or natural purification of streams and other bodies of water. In addition, however, sewage disposal by dilution is circumscribed by the wide use of water courses for purposes other than the reception, transmission and ultimate disposal of sewage. Lakes and streams are employed as sources of water supply for household and factory; they, as well as coastal waters, are centers of recreation for boating, bathing and fishing; like the fields they yield their crop of food stuffs to support useful animal life, such as fish, mollusks and crustaceans; finally they serve as waterways of commerce and for power development. The disposal of sewage into water, therefore, must be studied with due regard to the varied purposes that the water receiving the sewage may have to serve, and sewage treatment prior to disposal may become necessary not only to prevent water courses from being overworked as natural purifying agencies, but also to permit their economic employment for the other essential purposes which they may be called upon to serve.

Prior to 1890, little attention was given to objectionable conditions produced by the discharge of sewage into American rivers. With the subsequent rapid growth of cities and industries and consequent increase in water requirements, as well as sewage pollution, there has been a tendency towards greater restriction upon the discharge of untreated sewage into bodies of water, and towards raising the degree of purification required of treatment plants. While there can be little doubt that in certain cases progress in these directions has exceeded logical needs, it is equally true that over the greater part of the country the requirements are not sufficiently exacting. More care in disposing of

sewage will be necessary in the future, if the water resources are to be developed to best advantage. It is desirable, therefore, when adopting a plan for the treatment and disposal of sewage, to select such means as meet the needs of the present and immediate future, while lending themselves at the same time to extension into more effective treatment when it shall be required.

The limitations of sewage disposal by dilution may be summarized on the basis of the restriction of pollution called for by considerations of hygiene, esthetics, economics and law, as follows:

(*A*.) Hygienic Considerations.
- 1. Contamination[1] of
  - (*a*) Private and public water supplies.
  - (*b*) Natural ice.
  - (*c*) Shell fish.
  - (*d*) Waters at bathing beaches.
- 2. Pollution[1] resulting in
  - (*a*) Nuisance affecting the public health and comfort.
  - (*b*) Impairment of recreational facilities.

(*B*) Esthetic considerations, creation of conditions offensive to
- 1. The eye.
- 2. The sense of smell.

(*C*) Economic considerations, damage to
- 1. Industrial water supplies.
- 2. Live stock.
- 3. Fish and other useful aquatic life.
- 4. Property with resulting depreciation of values.
- 5. Private and public river and harbor improvements and navigation (such as silting due to sewage deposits).

(*D*) Legal considerations, interference with the rights of riparian owners.

272. Legal Aspects of Sewage Disposal.—The pollution of water has given rise to considerable litigation. In the case of unnavigable running streams, the law as interpreted in most states is based on the old common-law principles, (*a*) that every riparian owner is entitled to have the waters of the neighboring stream reach his property in their natural condition except for reasonable use of them by upper riparian owners, and (*b*) that every riparian owner is entitled to make such reasonable use of the water flowing past his property as he sees fit. In the semi-

[1] The American Public Health Association has defined *contamination* of water as the introduction into it of bacteria or other substances that tend to render it unsuitable for domestic use. *Pollution* of water is defined as the introduction into it of substances of such character and in such quantity as to render the body of water objectionable in appearance or to cause it to give off objectionable odors.

arid regions the legal doctrine of prior appropriation[1] prevails and court decisions regarding the pollution of waters in those regions have not been sufficiently important to show how far, if at all, the law which prevails in the Eastern States will be changed by the courts of final jurisdiction of the states where the common-law view of water rights does not prevail.

The two water rights of a riparian owner, under the common law, hinge on the interpretation of what is meant by a "reasonable" use of the water. For many years the courts have been engaged in settling suits involving that question, and today it is established in many states that a riparian owner can freely use the water for watering stock, household purposes and irrigation of land, provided these uses make no appreciable reduction in the volume of the stream and result in no pollution of its waters. It is evident, however, that what might be considered reasonable use of the stream for fishing, drainage of agricultural lands, removal of sawmill refuse and other purposes in sparsely settled regions would be prejudicial to public welfare in more populous districts. Hence the courts have adopted no rigid rules for interpreting the law, but have decided what was reasonable upon the merits of each case.

The principles of the common law also are applied to the waters of privately owned, natural ponds. If there are several owners, each has the same rights as the others, just as though the pond were a stream and the owner a riparian owner.

The invasion of public and private rights in ponds and non-navigable streams by turning the contents of sewers into them is not due to the collection of storm water in drains and sewers and its discharge into the natural drainage courses of the catchment areas. Storm water, no matter how much increased in volume by a reduction in the extent of permeable ground and changed in character by the inclusion of street refuse, is legitimately discharged into the streams naturally carrying away the run-off of the valley.

In the case of danger to public health, the common law, which is concerned with property rights, is supplemented best by statutes, as the extent to which the former will apply to infractions of sanitary principles is uncertain. The statutes may either be passed directly by a legislature, or the latter may clothe a

[1] The doctrine of prior appropriation is in force in Arizona, Idaho, Montana, Nevada, New Mexico, North Dakota and Wyoming, and in other states in the arid districts it is partly recognized.

commission with powers to make regulations which become, for all practical purposes, the same as statutes.

The legal aspect of water contamination is discussed in "Pollution of Inland Waters," by Edwin B. Goodell[1] where special attention is given to statutory laws on the subject. His conclusions regarding the rights and duties of municipal corporations are substantially as follows:

Considered as corporations, municipalities have only such rights and powers as are conferred on them by statute, either expressly or by necessary implication. When, under due authority, they become the owners of lakes, reservoirs and natural streams, they have the same rights to pure water, and are charged with the same duties, as other riparian proprietors. If authorized to construct systems of sewers draining into streams, such authority does not exempt them, except in Indiana, from the duty not to pollute the stream to the damage of lower proprietors. The common-law water rights of property owners cannot be taken from them for public use except upon payment of an amount determined by condemnation proceedings authorized by statute. Until municipal corporations have acquired by such proceedings the rights of all lower proprietors and paid for them, they are required in all cases to refrain from the pollution of streams, to the same extent as private owners.

NATURAL PURIFICATION ACCOMPANYING DILUTION

273. The Self-purification of Sewage-polluted Water.—Self-purification may be defined as "The natural process or combination of natural agencies that tends to render stable and innocuous foreign substances that find their way into water and so to restore the water to its natural condition of purity."[2] The forces of self-purification are operative in standing water as well as in running water. They have been studied more particularly in connection with sewage-polluted streams, because it is in such waters that the effects of self-purification or natural purification are most conspicuous and, therefore, defined most clearly. To understand the problems involved in the disposal of sewage by dilution, the student should be familiar with the mechanism of self-purification. This is a broad subject, worthy of more space than can be given to it in this chapter.

[1] Water Supply Paper 152, U.S. Geol. Survey, 1905.

[2] WHIPPLE: "Microscopy of Drinking Water," 4th ed., p. 313.

The processes of self-purification are physical, chemical and biological in nature. Like all processes involved in the purification of sewage, they are interrelated closely and mutually dependent, the resultant of their operations being a gradual return of polluted water to cleanly condition. The most important processes of self-purification are outlined below, together with a brief explanation of their operation.

(A) *Physical Processes.*

1. Dispersion.—Mixing of sewage with water results in the dispersion of the sewage matters, animate and inanimate. A dilution, for example, of raw sewage in 5 c.f.s. of water per 1,000 population signifies that the daily waste matters of one person are dispersed in about 3,300 gal. of water instead of in 100 gal. as in sewage. Improvement in appearance and odor and in concentration of possible pathogenic organisms is the natural result.

2. Sedimentation.—Heavy suspended particles settle out in standing water, slowly running water or in the back-waters of rapidly flowing streams. Light suspended particles and colloidal matters settle as aggregated or coagulated masses. While this removes these matters from the water, it transfers them to the bottom where they must be cared for as before. The formation of sludge deposits is often objectionable and intensifies the problems of pollution in certain localities, such as ferry slips, docks, or other reaches of quiescent water. This results in a type of decomposition which differs from that taking place in the water itself.

3. Light.—Sunlight is a disinfectant, destroying many objectionable bacteria. It is also a stimulant to the growth of green plants, large and small. These, by photosynthesis, give off oxygen and remove carbon dioxide. Sunlight also bleaches color.

4. Aeration.—As water comes into contact with the atmosphere an interchange of gases takes place whenever the concentration of the gases in the liquid and gaseous phases are not in equilibrium. This is particularly important in connection with the oxygen requirements of polluted water. As the oxygen dissolved in the water is used up by the decomposition of the organic sewage load, new volumes of oxygen are added by absorption from the atmosphere. This is called *reoxygenation* or more generally *reaeration.* Carbon dioxide and other gases of decomposition escape to the atmosphere.

(B) *Chemical Processes.*

1. Oxidation.—Aerobic organisms convert complex chemical substances into mineral matter, gases or relatively stable organic matter. Certain dissolved mineral constituents such as iron and manganese are oxidized and precipitated as insoluble compounds.

2. Reduction and Hydrolysis.—Anaerobic organisms liquefy and split the complex organic constituents of sewage and thus pave the way for stabilization by oxidation. In the complete absence of oxygen, offensive odors and gases are produced which escape to the air. Certain mineral substances are taken into solution. True anaerobic decomposition may

take place in the bottom sludge or ooze even when the supernatant water contains dissolved oxygen, because diffusion of oxygen is not sufficiently rapid to supply the deficiencies created in the bottom deposits.

3. Coagulation.—Mixing of certain wastes, especially industrial wastes, with sewage or other wastes sometimes results in chemical coagulation with consequent precipitation of dissolved, colloidal, and suspended matters.

4. Disinfection.—Certain poisonous substances, such as copper and arsenic, derived from industrial wastes, destroy bacteria and other living organisms. As a result they interfere with the biologically-activated purification processes until they are sufficiently diluted or removed by physical or chemical means to be reduced in concentration to the limits of tolerance of the living cells.

(*C*) *Biological Processes.*

1. Bacteria.—The food habits of these and other living reagents result in the decomposition and stabilization of the sewage matters by biochemical means. As the food substances are used up or the environment is unfitted for continued existence by the waste products of their activity, each group of organisms dies off and makes way for other groups which in turn flourish and succumb.

2. Plankton.—Plant forms (algae and fungi) utilize simple chemical substances resulting from bacterial activity. Green plants, by photosynthesis, give off oxygen while taking in carbon dioxide. Animal forms (protozoa, rotifera and crustacea) are scavengers and as such destroy organic substances much as the bacteria do. Some forms are predatory; protozoa ingest bacteria and are themselves consumed by crustacea. Rotifera feed on algae.

3. Larger Aquatic Life.—Rooted plants utilize food substances contained in bottom deposits. Worms "work over" sewage sludge. Insect larvae and other forms live on food substances in the water or bottom ooze. Fish may be scavengers or may live on plankton and insect larvae.

274. Progressive Changes in a Sewage-polluted Stream.—When sewage is discharged into water a progression of changes is set in motion which is manifested by changing physical, chemical and biological characteristics of the water. The events which take place are most clearly defined in a clean stream into which a large volume of sewage is discharged at one point. Discussion of this subject will therefore be hinged upon such a case.

Degradation.—The sewage, to begin with, renders the water turbid. Sunlight is shut out, and green plants, thus deprived of their energy for growth, soon die. The fresh organic matter is relished by some varieties of fish which, therefore, gather in the vicinity of the outfall. Decomposition of the organic matter, as the result of bacterial activity, soon begins and quickly reduces the available oxygen in the stream, making conditions unsuitable

for all forms of life except the most primitive organisms which subsist on decaying matter. These saprophytes,—bacteria, fungi and protozoa,—develop in tremendous numbers until one drop of the polluted water literally teems with microscopic life. The reduction in available oxygen is accompanied by an increase in the carbon dioxide content of the stream. Reaeration takes place, but is not able to keep pace with the deoxygenation. In all but rapidly flowing streams some of the suspended sewage matter settles to the bottom and forms sludge deposits or a *pollutional carpet* in which small reddish worms (*Limnodrilus* and *Tubifex*), together with other organisms, make their appearance. These "work over" the sewage sludge and are potent factors in its stabilization. The so-called sewage fungi (*Sphaerotilus natans*) also make their appearance near the outfall and are found in white bulbous masses on the bottom and attached to stones and sticks. During periods of high flow the sludge deposits are often washed away, and the stream is thus relieved of much of its sludge load. It is evident that the stretch of river immediately below the sewer out-fall is one in which life is reduced from a more highly organized to a more primitive plane, and the physical and chemical quality of the water is abased. Hence this first zone of pollution is referred to as the *zone of degradation.* Using the dissolved oxygen content of the stream as a criterion of its pollutional character, the zone of degradation occupies that river stretch immediately below the outfall in which the oxygen saturation is reduced to about 40 per cent.

Active Decomposition.—The zone of degradation is followed by a stretch of river in which the processes of decomposition which gradually establish themselves in the first zone become extremely active. For this reason it is called the *zone of active decomposition.* In this zone the water remains grayish in color. Decomposition of the complex organic materials is characterized by reducing and splitting processes. Soluble, volatile and gaseous compounds are formed. As long as the water contains dissolved oxygen no serious nuisance will result. In grossly polluted streams, however, the dissolved oxygen content may be completely exhausted in spite of the fact that reaeration increases as the oxygen content is reduced. Anaerobic or septic conditions will then prevail. During the summer months, foul odors are apt to arise, gas bubbles are evolved and the sticky blackened

sludge deposits may be lifted by the gases of decomposition into the water, turning the stream into a black putrefying water-course. As the available organic material is consumed, the bacteria and protozoa which have held sway die off and decomposition slackens. A point is reached when reaeration first balances and then overbalances the biochemical oxygen demand, and a new store of dissolved oxygen is gradually created. More highly organized life re-establishes itself. The trend of self-purification becomes positive rather than negative as heretofore. The dissolved oxygen saturation first falls below 40 per cent, sometimes to zero, and then gradually climbs back to above 40 per cent.

Recovery.—There follows the zone of active decomposition a third zone in which the stream gradually recovers its former appearance and normal condition. The water becomes clearer; algae and larger aquatic vegetation reappear. The dissolved oxygen content is slowly increased to full saturation by reaeration and usually to a less extent by the photosynthetic activities of green plants. Small animal forms of life re-establish themselves, and some of these serve as food for fish. Judged by chemical standards this *zone of recovery* is one of mineralization. Nitrogen end products are carried to nitrites and finally to nitrates, sulphur to sulphates and carbon to carbon dioxide or carbonates.

Cleaner Water.—Below these three zones of turbulent self-purification is found the *zone of cleaner water* in which natural and desirable stream conditions are once more established. The water becomes attractive in appearance, but may still harbor pathogenic organisms. Long periods of flow are often required to destroy these.

Some of the progressive changes which take place in a sewage-polluted stream are exemplified by the results of a study made by Whipple in 1912 of the Genesee River, into which the sewage of Rochester was discharged at that time (see Table 73). The succession of life, the deoxygenation and reaeration of the waters, the rise and fall in carbon dioxide content and the mineralization of the organic matter are clearly defined. Self-purification in this case, however, was hastened by the diluting effects of Lake Ontario into which the river discharges 5.5 miles below the sewers. Far longer river stretches are commonly required to bring about the purification shown.

TABLE 73.—CHEMICAL AND BIOLOGICAL CHANGES IN GENESEE RIVER BELOW THE SEWERS OF ROCHESTER, 1912
(After Whipple)

Miles below sewers	Organisms per cc.					Dissolved oxygen, per cent saturation	Carbon dioxide, p.p.m.	Nitrogen,[1] p.p.m.			
	Bacteria	B. coli	Protozoa	Algae	Rotifera and Crustacea			Organic	Ammonia	Nitrite	Nitrate
0.0	*2,040,000*	*10,000*	94	178	150	34.5	15.5				
0.5	....	......	.....	...	...	16.2	22.5				
1.5	713,000	1,000	*2,836*	441	67	7.7	29				
2.5	....	......	.....	...	...	4.2	*31*				
3.5	8,000	......	501	*507*	100	*2.8*	26	0.84	1.26	0.014	0.00
4.5	....	50	.....	...	...	16.9	21				
5.0	....	......	330	315	*850*	24.4	19.3				
5.5	4,000	10	110	252	100	34.1	13	0.23	0.78	0.006	0.05

[1] Determinations of August 13; other determinations of July 24, 1912.

The zones of pollution and natural purification neither occupy a fixed position in streams nor are they sharply bounded. They shift with changes in temperature and variations in river flow and waste discharge. During the summer, when life and with it decomposition are most active, the zones are more pronounced. Depending upon the load of sewage which the stream receives, the zones occupy longer or shorter river stretches. In slightly polluted streams, the zone of active decomposition may be suppressed; with heavy pollution septic conditions may prevail for many miles. The progress of self-purification may be interrupted or accelerated by the discharge of new wastes on the one hand or by dilution due to the entrance of cleaner water on the other. In lakes and tidal estuaries pollution and self-purification are influenced by winds and by tides or other currents. Bottom deposits, being less subject to the changes which may occur in the water, are more representative of average conditions and are therefore an especially useful guide in pollution studies.

275. The Oxygen Balance.—The importance of the oxygen conditions existing in polluted waters, as reflecting as well as deciding the nature of the progress of decomposition of sewage which is disposed of into water, must be evident from the foregoing discussion. The relation between the biochemical oxygen demand of the sewage and the oxygen available in the diluting water is called the *oxygen balance*. Since no nuisance ordinarily will result as long as there is sufficient oxygen present to meet the demand, the oxygen balance is probably the best measure of the amount of sewage which a given water course can handle without the creation of a nuisance.

The following factors determine the oxygen balance:

1. The biochemical oxygen demand of the sewage as discharged, which at times must be considered to be composed of
   (*a*) The demand of the liquid.
   (*b*) The demand of the suspended solids.
   (*c*) The demand of the deposited sludge.
2. The biochemical oxygen demand of the diluting waters. This is sometimes quite appreciable, due to prior pollution by sewage or by industrial wastes.
3. The dissolved-oxygen content of the sewage at the time of discharge.
4. The dissolved-oxygen content of the diluting water.
5. The oxygen absorbed by reaeration from the atmosphere.
6. The oxygen given off by green plants.
7. The oxygen readily available from mineral matter such as nitrites and nitrates.

Determinations of the biochemical oxygen demand and the oxygen available in the sewage and water have been described in Chapter XI, together with the formulation of the *deoxygenation curve.* Reaeration, however, has only been lightly touched upon. Two types of reaeration may be distinguished, atmospheric and photosynthetic. Of these the former has been formulated mathematically and is of more general applicability; the latter depends so much on local conditions that formulation and prediction of its effects are not readily made.

276. Reaeration.—An example of the reoxygenation of polluted water by green plants, particularly the plankton, is afforded by the authors' observations of a small stream at Cincinnati, O. A sewerage system serving a population of about 1,700 persons discharged sewage into the upper reaches of this stream, which had a sluggish flow for about a mile below the sewer outlet and then spread out into a shallow creek with occasional pools. The odors from the water just below the sewer outlet were offensive at times, causing much complaint. At the time of the inspection, the water near the outlet contained about 70 per cent of its saturation value of dissolved oxygen. Farther downstream green algae appeared and their growth increased in luxuriance as the distance from the outlet increased. Where it was heaviest, there were evolved by the green masses enormous numbers of gas bubbles which, when collected in a bottle and tested by ignition, appeared to be pure oxygen. At a distance of 1¾ miles below the outlet the water was 130 per cent saturated with oxygen and free from objectionable appearance. Aquatic growths of this type are active both in the reaeration of polluted water and in the sedimentation and straining out of suspended material.

Observations similar to these have been made at other places. The most notable ones are those of the United States Public Health Service on the Potomac River flats below Washington, D. C. Of special interest in this investigation are the studies of the effect of sunlight on plant activities. Purdy found that the average saturation with dissolved oxygen of the water flowing from the flats was 99 per cent on sunny days and only 69 per cent when the sky was cloudy or overcast.

The amount of oxygen absorbed by water from the air in a unit of time is dependent upon the following factors:

1. The degree of undersaturation of the water.
2. The area of water surface exposed per unit volume.

3. The agitation of the water surface by wind, currents, and other disturbances such as those due to rapids, dams and ships' propellers.
4. The downward diffusion of the oxygen.

Reaeration, therefore, varies greatly in different bodies of water. In a turbulent stream the rate of reaeration is high because new surfaces of water are being brought constantly into contact with the atmosphere. In a calm deep lake the rate is relatively low because in the absence of agitation the lower layers of water are dependent for reaeration upon the diffusion of oxygen from the upper ones. This is a slow process.

Since the rate of solution of a gas in water varies directly as the degree of undersaturation, the effect of the first factor on reaeration can be formulated as follows:

Let D = initial oxygen saturation deficit in p.p.m.
D_t = oxygen deficit at any time t in p.p.m.
K_2 = reaeration constant

Then following the development of the *deoxygenation curve* (Section 265), $\log \frac{D_t}{D} = -K_2 t$

K_2 cannot be determined in the laboratory as was K_1, but varies with the body of water under consideration. It takes into account the other factors listed above. In the studies of the Ohio River the United States Public Health Service found K_2 to vary with stream depth and various physical conditions affecting turbulence of flow. This was expressed by:

$$K_2 = CV^nH^{-2}$$

where

V = velocity of flow in ft. per sec.
H = depth of flow in ft.
C and n = river constants.

Diffusion being affected by viscosity, which in turn is dependent upon temperature, it is possible to make an allowance for variation in temperature. It has been determined to vary as follows: K_2 at any temperature T equals K_2 at 20°C. times (1.0159^{T-20}).

The magnitude of the reaeration constant has been determined by the U. S. Public Health Service for certain stretches of the Ohio and Illinois Rivers. In the normal stretches of flow of these rivers the average value of K_2 was 0.24. In the shallow rapids of the Des Plaines River below Joliet, Ill., where the channel is steep and rough, however, the average value of K_2 was

2.00. Further work of this type is much needed for streams as well as lakes and tidal estuaries. In quiet water the value of K_2 at any temperature varies inversely with some power of the depth and directly with the amount of mixing that takes place. A study of experiments by Adeney yields a value of 0.115 for K_2 at 20°C. for each foot depth of water and $K_{2(T)}$ is found to equal $K_{2(20)}$ (1.020^{T-20}).

277. The Oxygen Sag.—Deoxygenation and reaeration combine to produce in streams dissolved oxygen values which, when plotted as ordinates with time of flow below the point of sewage discharge as abscissas, yield a curve characteristic of

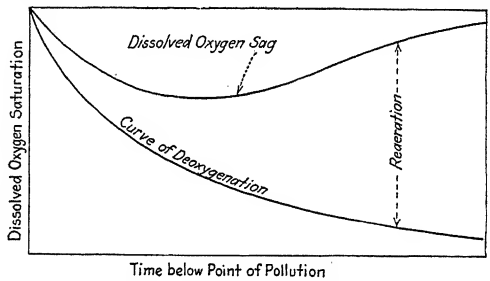


FIG. 131.—The dissolved oxygen sag.

self-purification and known as the *dissolved oxygen sag*. The equation of this curve is given by combining the equations of deoxygenation and atmospheric reaeration, obtaining the following formula:[1]

$$D_t = \frac{K_1 L}{K_2 - K_1}(10^{-K_1 t} - 10^{-K_2 t}) + D \times 10^{-K_2 t},$$

where

D_t = oxygen saturation deficit of the water in p.p.m. at any time t

D = initial oxygen saturation deficit at point of pollution, in p.p.m.

L = initial biochemical oxygen demand of the diluted sewage, in p.p.m.

K_1 = the deoxygenation constant

K_2 = the reaeration constant.

The general outline of the oxygen sag is shown in Fig. 131 together with the oxygen conditions which would obtain in the absence of reaeration.

[1] For derivation of this formula see Appendix A, Public Health Bull. **146,** U. S. Public Health Service.

In computing the probable dissolved oxygen sag of a stream into which sewage is discharged, the following factors must be known or estimated.

(*A*) For the sewage, assuming that it is fresh:

1. Quantity
2. Temperature
3. Dissolved oxygen content
4. Biochemical oxygen demand during the first stage (commonly computed from the 5-day demand).
5. Deoxygenation constant.

(*B*) For the water, assuming that its own biochemical oxygen demand falls within the first stage:

1. Quantity
2. Temperature
3. Dissolved oxygen content
4. Biochemical oxygen demand
5. Reaeration constant.

Example.—Find the oxygen sag for a clean stream discharging 40 c.f.s. into which 1 m.g.d. of fresh sewage is emptied. Assume that the 5-day, 20°C., B.O.D. of the sewage is 200 p.p.m. and that it contains 6.00 p.p.m. of oxygen. Assume further that the temperature of both the water and sewage is 25°C. and that the water is saturated with oxygen above the point of pollution, but has a 5-day, 20°C., B.O.D of 1 p.p.m. Assume that the deoxygenation constant of the sewage at 20°C. is 0.10 and that the reaeration constant of the stream at 20°C. is 0.24.

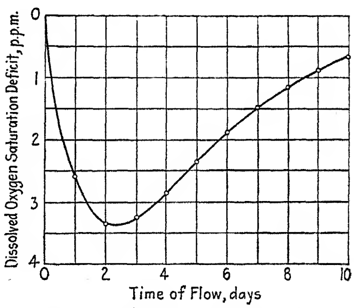


FIG. 132.—Dissolved oxygen deficit of Example in Section 277.

In the oxygen sag formula

$$K_1 = 0.10 \times 1.047^{25-20} = 0.126$$
$$K_2 = 0.24 \times 1.016^{25-20} = 0.26$$
$$D = 8.35 - \frac{(8.35 \times 40 + 6.00 \times 1.547)}{41.547} = 0.09 \text{ p.p.m.}$$
$$L = \frac{1.00}{0.68}\left(\frac{1 \times 40 + 200 \times 1.547}{41.547}\right)[1 + 0.02(25 - 20)] = 13.60 \text{ p.p.m.}$$

The resulting curve is shown in Fig. 132.

The student should familiarize himself with the various forms which the oxygen sag may take, depending upon the factors entering into the problem. He should remember that the theoretical formula applies only to the first stage of deoxy-

genation and that, in streams, conditions may be profoundly modified by the presence of other wastes, the entrance of diluting waters or additional wastes, green plants, dams and rapids, and many other factors. To illustrate some of these modifications, Fig. 133 is given. It shows the dissolved oxygen sag of the Skaneateles outlet in central New York as affected by a number of wastes which enter the stream along its course.

If the velocity of flow of a stream is known, the oxygen sag can be determined with respect to distance below the point of pollution. The average velocity naturally varies considerably. In the Ohio River studies previously referred to, monthly mean

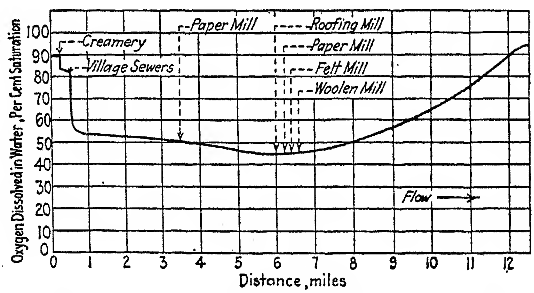


FIG. 133.—Dissolved oxygen sag of Skaneateles outlet. (*After Suter.*)

velocities ranging from 0.16 to 3.60 miles per hour were encountered in 1914–15 between Pittsburgh, Pa., and Louisville, Ky. An average value for velocity of flow of large rivers is ⅔ mile per hour.

278. Temperature Effects.—Of the various factors which must be considered in the oxygen balance, temperature is one of very general significance. Both deoxygenation and reaeration increase with rising temperatures and decrease with falling ones, the former more so than the latter. This, as shown in Fig. 134, is reflected in the shape of the oxygen sag.

At low temperatures the sag is flat, but long; at high ones it is deep, but short. Taking into account the fact that at high temperatures water can carry less oxygen in solution, it follows that septic conditions are more likely to prevail during warm weather. During cold weather, on the other hand, self-purification is evidently retarded and the zones of self-purification extend over longer river stretches. In the disposal of sewage by dilution, therefore, rivers, lakes or tidal estuaries may receive in

winter, without causing objectionable conditions, quantities of sewage which in summer would give rise to nuisance.

Bacterial action at low temperatures is relatively slow, and sewage may be carried by the water receiving it to a point where ample dilution is afforded before the bacteria have caused any offensive conditions. This has an important economic aspect, for, in summer, sewage treatment may be carried to a degree insuring satisfactory conditions at an expense which, if continued through the year, would be prohibitive. Advantage may be taken of the winter conditions by providing a less expensive treatment, care being taken to avoid the formation of sludge deposits which may prove objectionable during the succeeding warm season (see Section 282).

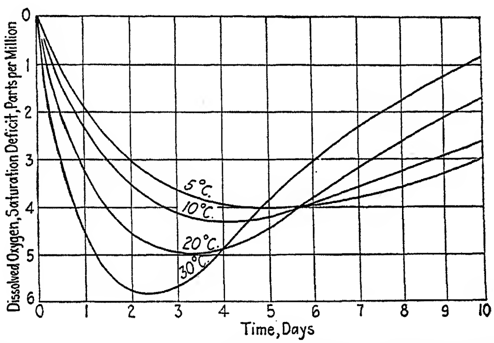


FIG. 134.—Effect of temperature variations on the progressive changes in the dissolved oxygen deficit. $L = 20$ p.p.m. $D = 0$. (*After Streeter.*)

When freezing temperatures are reached and streams become covered with ice, reaeration is prevented and offensive conditions may obtain. This factor may be of considerable importance in heavily polluted waters in which the oxygen in the waters below the ice sheet may be depleted completely.

279. Salt Water and Fresh Water.—The rate of deoxygenation of sewage in salt water during the first stage of decomposition seems to be much the same as that in fresh water. Experiments show it to be only slightly lower. The so-called nitrification stage, however, seems to be delayed. Reaeration, too, is much the same as in fresh water. As far as immediate oxygen demands are concerned, therefore, the disposal of sewage into salt water appears to depend upon the same factors that play a part in fresh water, with the exception that salt water will hold in solution

about 20 per cent less oxygen. For this reason fresh water should be able to dispose of more sewage than is salt water, other things being equal. More experience must be gained before it can be said whether or not conditions are equal.

The salinity of sea or brackish water can be measured either by determining its specific gravity or its chloride content. The specific gravity of normal sea water is about 1.025, corresponding to a chloride content of 18,000 p.p.m. The relative proportion of sea water in a tidal estuary is therefore given by the ratio of the specific gravity to 1.025 or the ratio of the chloride content to 18,000. Specific gravity is commonly determined by means of a *salinometer*, a special form of hydrometer.

When sewage is discharged into salt water there is a greater tendency for the suspended solids to settle and to form sludge banks than when it is discharged into fresh water. This is due in part to the coagulation of the finer suspended solids by the sea salts. The sludge banks, however, seem to use up oxygen at a lower rate than do sludge banks in fresh water (see Section 282).

280. Tides and the Tidal Prism.—If a river discharges into a tidal estuary, a mixture of fresh and salt water is produced, changing in character from hour to hour. If the estuary is short and steep, it clears itself with each ebb tide; if long and with complex entries, the water may oscillate backward and forward with a varying degree of salinity. Inasmuch as the tidal currents must be utilized to remove sewage, it is necessary to have a comprehensive knowledge of them before locating the outlet of a sewer system, in order to be sure that offensive conditions will not arise at any stage of the tide, and that bathing beaches and shellfish layings will not become polluted.

At the turn of the tide, the incoming salt water follows the bottom, owing to its greater density, while the overlying brackish water is still traveling seaward. Gradually, as the depth of the incoming wave increases, the motion of the entire section changes, flowing landward with increasing velocity until the maximum velocity, or *strength of the tide*, is reached. The velocity then falls off until ebb tide begins, when the whole mass of water flows seaward with increasing velocity until the maximum run is reached. The velocity then decreases until the turn of the tide is reached and the cycle begins again. The velocity of the ebb tide is greater than that of the flood tide because the flow of fresh

water is in the same direction as that of the outgoing salt water, instead of in the opposite direction, as on the flood tide.

The *tidal prism* of a tidal basin is the volume of water contained within it between the elevations of high and low water. It depends upon the mean range of the tidal rise and fall, which generally decreases in going from the northern to the southern limits of the United States, as follows:

East Coast		West Coast	
City	Range, ft.	City	Range, ft.
Boston	9.6	Seattle	11.6
New York	4.4	Astoria	6.4
Baltimore	1.2	San Francisco	4.0
Charleston	5.2	San Diego	4.0
Savannah	6.5		
Key West	1.2		
Galveston	1.0		

In order to find the amount of diluting water the volume of the tidal prism must be computed. An example of the necessary calculations follows.

Example.—Find the dilution afforded a sewage flow of 2.3 m.g.d., which is discharged at the upper end of a tidal estuary with a surface area of 50 million sq. ft. at mean low water and 53 million sq. ft. at mean high water, the mean range of tide being 9.5 ft. Float observations show that three successive ebb tides are required to carry the sewage seaward from the outfall to a point from which it will not return to the estuary. The average cycle of tides occupies a time of 12 hours 20 min.

$$\text{Volume of tidal prism} = \frac{50 + 53}{2} \times 1{,}000{,}000 \times 9.5 \times 7.48$$

$$= 3{,}660 \text{ million gal.}$$

$$\text{Volume of sewage} = \frac{2.3 \times 3 \times 12.33}{24} = 3.55 \text{ million gal.}$$

$$\text{Dilution} = \frac{3.55}{3{,}660} = 1\!:\!1{,}030.$$

Observations of the movements of floats are made to ascertain the direction of currents. In making use of floats to determine the direction and velocity of tidal currents it must be kept in mind that sewage discharged into tidal water will tend to rise to the surface and will then move over the surface more rapidly than will deep floats. For example, the early investigations of the tidal

currents in the vicinity of the Moon Island outlet of the Boston sewerage system were made with floats 8 ft. long. These floats showed the direction of the currents, but they moved with the mean velocity of the water and not with the surface velocity during the ebb tide, during which alone it was proposed to discharge the sewage. After this outlet was placed in service it was found that the sewage moved on the surface much more rapidly than was anticipated from the results of the float observations, and in later observations of this nature the Massachusetts State Board of Health used floats about 8 in. long for most observations and 24 in. long for the remainder, as it was found that the sewage tended to form in a shallow sheet on top of the water, and hence the surface and shallow depth velocities were of greater importance than the mean velocities of deeper sections from the surface down.

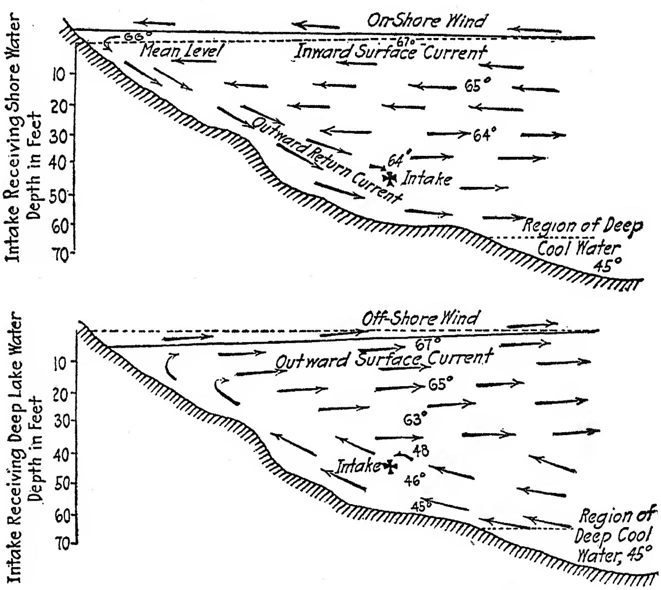


FIG. 135.—Effect of wind on lake currents at Milwaukee.

281. Wind Effect.—The wind blowing over the surface of a body of water produces waves and tends to set up currents in it, owing to the frictional resistance presented by the water to the currents of air over its surface. On-shore and off-shore winds, by piling up the water along the shore or carrying it

away from the shore, induce complementary currents in opposite directions at greater depths (see Fig. 135).

On the Great Lakes, surface currents have been found to travel at 3 to 6 per cent of the wind velocity, and the U. S. Hydrographic Service has reported[1] that a study of many observations in the North Atlantic indicated surface currents of about 3.2 per cent of the velocity of the wind. It is believed that the effect of these surface currents may extend to depths of 30 or 40 ft. Observations on the Great Lakes have shown that sludge deposits on the shores at these depths have been removed by scour. In tidal basins the scouring action of the currents are at times sufficiently reinforced by the wind to carry away sludge banks which otherwise would not have been moved.

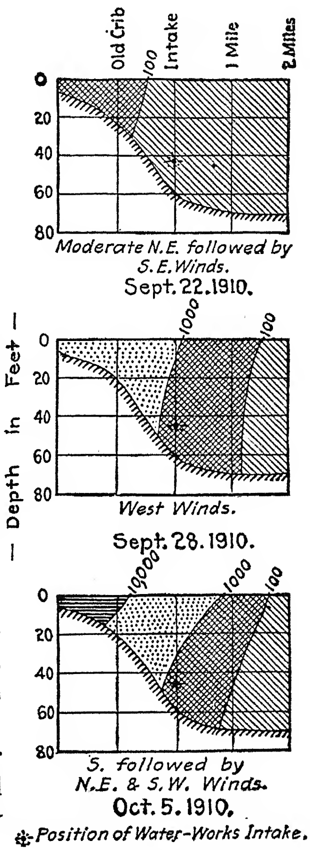


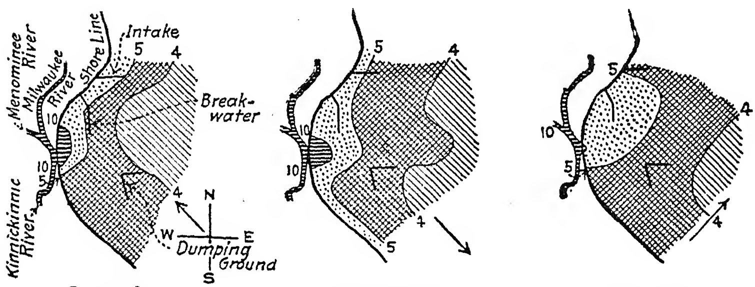
FIG. 136.—Effect of winds on vertical distribution of bacteria in a plane passing east and west through the intake of the Milwaukee water-works.

Wind action is also of importance in connection with reaeration. Wave formation increases the surface area through which oxygen can be absorbed from the atmosphere. This is particularly marked when the wind is strong enough to form white caps and the water is sprayed into the air. Diffusion of oxygen from the upper to the lower water strata is increased by the vertical currents induced by the wind.

Knowledge of the vertical currents produced by the wind has been gained largely by studies of temperature changes in the water at different depths. A study of these currents is of particular moment in the location of sewage outfalls with respect to water supply intakes and bathing beaches on inland lakes.

[1] *Monthly Weather Review*, 1902.

In 1909 to 1911 an investigation was made by Alvord, Whipple and Eddy of the methods of disposing of sewage and protecting the water supply of Milwaukee. This involved a study of the currents in Lake Michigan, which exemplifies the scope of such investigations. As the capacity of the lake is about equal to its discharge during 100 years, the lacustrine current toward its outlet at the Straits of Mackinac is inappreciable, and temperature and winds are responsible for both surface and deep movements of the water.



Sept.22.1910.
Wind.
Sept. 20. N.E.& S.W. Light
" 21. N.E. Moderate
" 22. S.E. Light

Sept.28.1910.
Wind.
Sept. 26. N.E. Brisk
" 27. W. Light
" 28. N.W. Light

Oct.5.1910.
Wind.
Oct. 3. S. Brisk
" 4. N.E. Light
" 5. S.W. Brisk

FIG. 137.—Effect of winds on distribution of chlorides in surface waters of Milwaukee Bay.

Fig. 135 illustrates the manner in which temperature records define the cause of wind-induced shore currents. An on-shore wind carries surface waters to the intake; an off-shore wind brings in the lower water strata. The quality of these waters depends upon the location of the source of pollution as shown in Figs. 136, 137, and 138. Thermophone[1] readings showed that the water below a depth of about 65 ft. is practically stagnant in the summer. Nearer the surface, however, the wind has a marked effect in circulating the water.

The temperature observations were supplemented by investigations of the number of bacteria in samples of water collected at numerous points, at both the surface and the bottom, and by determinations of the chlorides in the water at the same points. The results obtained on two days are shown in Figs. 136 and 137,

[1] The thermophone is an electrical resistance thermometer in which a current interruptor and telephone permit the determination of the temperature. For a description of this instrument and its uses see WHIPPLE, "Microscopy of Drinking Water."

and illustrate how the wind affected the distribution of the sewage through the lake in the vicinity of the city.

On Sept. 22 there was an on-shore wind. The bottom water had practically the same temperature as that at the surface and the turbidity of the water was the lowest observed at any time during the investigation. The surface water containing the largest amount of chlorides was driven to the northward and kept near the shore and the largest number of bacteria was found near the shore. The sewage of the city was discharged at that time into the lake by the rivers shown in Fig. 137.

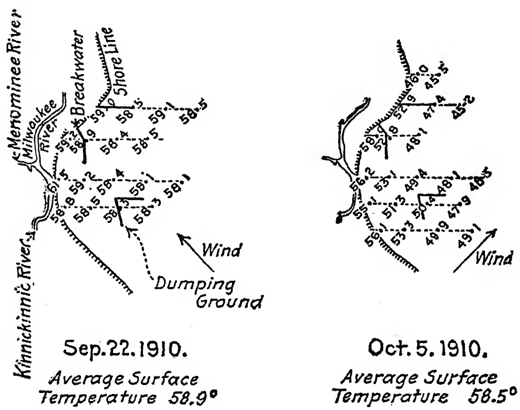


FIG. 138.—Effect of winds on temperatures of bottom waters in Milwaukee Bay.

On Oct. 5, the wind was off-shore and colder water was brought in at the bottom, the difference between the average surface temperature and the lowest bottom temperature being 13.3°F., as is evident from Fig. 138. The surface water containing the largest amount of chlorides had been driven to the north and there was a well-marked surface drift of water with large numbers of bacteria toward the water works intake. The importance of a study of prevailing winds and the currents produced by them is evident.

282. Sludge Deposits.—The settling of sewage solids in streams, lakes and tidal estuaries, while an aid to the clarifying of their waters, may intensify the pollution problems and create local nuisances wherever sludge banks are formed. Sludge deposits decompose more slowly than do the transported solids, but whereas the latter are removed to points of greater dilution, the settled sewage matters remain until scoured away in rivers by

floods, in lakes by wind-induced bottom currents, and in the sea by tides and winds. In warm weather gases and offensive odors may be given off from sludge banks in spite of the fact that the supernatant waters contain oxygen. Oxidation takes place only at the surface of the sludge. Truly anaerobic conditions prevail within the sludge masses. During the winter, decomposition is retarded, and unless removed by spring freshets, tides and high winds, the winter's accumulations may be left to putrefy during the summer. The Sewage Disposal Committee of the Chicago Board of Review, for example, estimated that during the summer the cumulative effect of sludge deposits on the total oxygen demand in the lower stretches of the Illinois River amounted to 0.282 lb. per capita per day. This value is slightly greater than the estimated B.O.D. of the sewage itself (0.24 lb. per capita per day), so that during the summer the oxygen requirements of the river would be doubled or more than doubled. While these figures apply only to this particular stream, the value of removing settling solids from sewage becomes apparent.

In salt water decomposition of sludge deposits does not appear to proceed as violently as in fresh water.

283. Bacterial Death Rates in Polluted Waters.—From the hygienic standpoint, the fate of the pathogenic organisms which may find their way into the sewage and thence into streams and other bodies of water is the most important consideration in sewage disposal by dilution. Whipple has estimated that *Bacillus typhosus*, the causative organism of typhoid fever, will die away in water at a more or less uniform rate, 50 per cent being destroyed in from 1 to 3 days and 90 per cent in from 3 to 13 days. Some of the more resistant organisms may live for many weeks and may be carried possibly 80 miles or more, still retaining their power of infection. As previously stated, they die more rapidly in heavily polluted and warm water than in clean and cold water. It has been found that the viability of this organism is very similar to that of *B. coli*. For this reason a statement of the death rate of this *index organism* of pollution will yield information on what happens to the pathogenic bacteria. The results of studies by the U. S. Public Health Service on the Ohio River below Cincinnati are shown in Table 74.

The death rates of bacteria growing on agar and gelatin are shown as well as those of *B. coli*, in order to permit a comparison of their relative significance.

TABLE 74.—BACTERIA REMAINING BELOW THE SEWER OUTFALLS OF THE CINCINNATI METROPOLITAN DISTRICT

Mean gage height, ft.	Mean time from sewer outlets, hours	Reduced to basis of 100 per cent[1] at maximum		
		Agar, 37°C.	Gelatin, 20°C.	*B. coli*
24.9	3.2	93.52	91.79	82.36
21.2	7.2	79.04	80.32	84.50
10.6	11.9	96.95	99.87	100.00
12.1	17.4	100.00	100.00	91.83
11.4	25.2	66.65	81.08	67.45
16.1	35.3	49.05	77.41	36.83
9.8	44.8	29.96	54.35	37.67
9.7	60.5	18.82	29.95	16.41
9.8	83.4	9.43	13.66	10.99
7.0	112.6	3.71	4.82	5.42
6.1	139.9	1.34	1.59	1.23
4.3	182.9	0.68	1.12	0.380
4.4	219.2	0.58	0.95	0.122
3.4	258.2	0.21	0.38	0.174
2.5	314.8	0.070	0.104	0.175
1.3	497.7	0.063	0.100	0.065

[1] Figures noted in these columns are in per cent of the numbers observed in the zone of maximum pollution below the district, April to November, inclusive, 1914, 1915, 1916.

DILUTION REQUIREMENTS

284. Standards of Dilution.—As stated in Section 271, the dilution method of sewage disposal is limited in any particular case by the uses to which the water course receiving the sewage is to be put. Standards of dilution, therefore, must be set with this in mind. Where the available dilution is small the standards must include standards of sewage treatment. In the United States no general standards of efficiency of sewage disposal have as yet been adopted. There has been amassed, however, as a result of practical experience and research, certain information which will serve as a guide in problems of disposal by dilution. This information relates in particular to the following:

1. The prevention of nuisance.
2. The safeguarding of water supplies.
3. The protection of bathing beaches.
4. The conservation of useful aquatic life.

In the British Isles, where rivers are small and their catchment areas are relatively densely populated, the Royal Commission fixed upon certain standards for sewages and sewage effluents which are outlined below in tabular form:

TABLE 75.—CONCLUSIONS OF THE ROYAL COMMISSION ON SEWAGE DISPOSAL, 1912

	Required characteristics of sewage effluent		Type of treatment that will satisfy the standards
	5-day B.O.D., 18.3°C., p.p.m.	Suspended solids, p.p.m.	
General standard....	≦20	≦ 30	Complete treatment
Special standards where sewages are diluted with the following volumes of water			
150–300..........	....	≦ 60	Chemical precipitation
300–500..........	....	≦150	Plain sedimentation
over 500.........	....	.....	No treatment required

English sewages are two to four times as concentrated as American sewages.

285. Prevention of Nuisance.—In the statement of the dilution required for the prevention of nuisance, recourse is commonly taken in American practice to the investigations and expressed opinions of recognized authorities in the field. The most important of these are summarized below:

In 1887, Hering, Williams and Artingstall recommended that the Chicago Drainage Canal be designed to provide 4 c.f.s. of lake water per 1,000 persons. In the adopted design this value was later reduced to 3.33, a figure that experience showed to be too low, due in part, at least, to the fact that the population equivalent of the industrial wastes was not considered and sludge deposits made large demands upon the diluting waters (see Section 282).

After an investigation for the Massachusetts State Board of Health,[1] Frederic P. Stearns, then its Chief Engineer, concluded that if the flow is less than 2½ c.f.s. per 1,000 inhabitants an offense would be almost sure to

[1] Special Report on Water, 1890, p. 791.

arise.[1] With larger volumes than 7 c.f.s. per 1,000 inhabitants, the pollution would be too small to cause any nuisance.

In 1897 the Ohio State Board of Health had an investigation made of the condition of certain Ohio rivers under the direction of Allen Hazen.[2] In discussing the results he stated that in the case of sluggish streams, or of streams the waters of which are already somewhat polluted, the quantity required for proper dilution may become 6, 8 or even 10 c.f.s. per 1,000 population.

In 1902 another investigation was made for the Massachusetts State Board of Health by Goodnough,[3] who succeeded Stearns as Chief Engineer of the Board. He narrowed the range of dilution fixed by Stearns and summarized the results as showing that where the quantity of water available for the dilution of the sewage in a stream exceeds about 6 c.f.s. per 1,000 persons contributing sewage, objectionable conditions are unlikely to result. Where sewage is discharged at many outlets into a large body of water he thought that objectionable conditions might not result from somewhat less dilution than that named. In every case where the flow was less than 3.5 c.f.s. per 1,000 persons objectionable conditions had resulted.

In 1902, in a letter to the Committee on Charles River Dam, Goodnough[4] made the following statement:

"The degree of dilution which has been found necessary to prevent unsanitary conditions where sewage is discharged into a stream, assuming 75 gal. of sewage per person, ranges between 20 to 1 and 60 to 1. In estimating the degree of dilution of the sewage, no account has been taken of the purifying effect of the water of the basin itself."

J. Herbert Shedd[5] testified in 1902 before the Commiteee on Charles River Dam that about 5 c.f.s. of the ordinary flow of the river, per 1,000 persons contributing sewage, would render the presence of the sewage unobjectionable.

In 1908 and 1913 Goodnough made for the Board investigations of the condition of the Merrimack River, the results appearing in special reports. The condition of other rivers was examined to obtain corroborative evidence regarding pollution, and it was found that wherever the dilution was 3.4 c.f.s. or less per 1,000 persons, a nuisance followed. Where serious pollution was observed with higher rates of dilution the nuisance was usually due to the discharge of large quantities of industrial wastes into the stream.

The Metropolitan Sewerage Commission of New York estimated in its 1910 report that the sewage discharged into New York Harbor was diluted with 32 parts of water, and that this ratio would become 1 to 13 about 1940. The flow of diluting water in the harbor was estimated at 4.7 c.f.s. per 1,000 population, which would be reduced to 2.65 c.f.s. in 1940.

Dilution factors of 3.5 to 6 c.f.s. per 1,000 persons are used as common guides by engineers. They represent at best very rough

[1] Stearns estimated that each person contributed to sewage daily an average of 0.015 lb. free ammonia, 0.003 lb. albuminoid ammonia, 0.218 lb. dissolved solids and 0.042 lb. chlorides.

[2] Preliminary Report of an Investigation of Rivers, p. 32.

[3] Report, State Board of Health, 1902, p. 452.

[4] Evidence, etc., before Committee, p. 108.

[5] Evidence, etc., before Committee, p. 365.

measures of the amount of dilution required and must be modified by considerations of prior pollution, industrial wastes, sludge deposits and successive increase in pollution or dilution. The American equivalent of the dilution ratio of 500:1, suggested for raw sewage by the Royal Commission, corresponds to a flow of 27 c.f.s. per 1,000 population. This standard evidently takes into account a relatively high use of the streams by successive municipalities and industrial establishments.

As more is learned about the reaeration characteristics of water it will be possible to come closer to the true limits of dilution required with and without treatment. The principle of regulating dilution on the basis of oxygen demand and supply was clearly recognized by the Engineering Board of Review of the Sanitary District of Chicago. Wherever possible this newer approach to the problem should be taken.

286. Safeguarding of Water Supplies.—In 1914 the International Joint Commission on the Pollution of the Boundary Waters between the United States and Canada concluded among other things that, in general, protection of public water supplies is more economically secured by water purification at the intake than by sewage treatment at the sewer outlet, but that under certain conditions both water purification and sewage treatment may be necessary. The Commission was unwilling to fix more than a tentative standard of 500 *B. coli* per 100 cc. of water as the limit of safe loading of water-purification plants. Studies by the United States Public Health Service have since yielded valuable information on the permissible loading of water purification plants drawing upon polluted sources of water. Using bacteriological standards, it was found that the purification accomplished could be expressed by the following formula:

$$E = CR^n,$$

where

E = effluent count
R = raw water count

C and n = purification constants, C measuring the magnitude of efficiency and n the relative constancy in bacterial quality.

The numerical values of C and n for *B. coli* were found to be 0.15 and 0.22 respectively, for the chlorinated effluent of 10 Ohio River plants studied in 1923–24. Substituting these values in

the above formula the permissible number of *B. coli* in the raw water is 5,000 per 100 cc. in order that purified chlorinated water may meet the revised "Treasury Standards" of 1 per 100 cc.

Subsequent studies on an experimental plant at Cincinnati have yielded values of C and n of 0.0008 and 0.82, respectively, for purified and chlorinated effluents, and of 0.029 and 0.77 for filtered but unchlorinated effluents. These values indicate permissible counts of 100 *B. coli* per 100 cc. of raw water for unchlorinated mechanical filter-plant effluents and of 6,000 *B. coli* per 100 cc. of raw water for chlorinated filter plant effluents. With the aid of the die-away curves of *B. coli* in polluted waters, it is therefore possible to estimate the dilution and treatment of sewage required to safeguard water supplies. Only in relatively lightly polluted waters in which long periods of flow, high dilution or great storage permit self-purification to be carried to complete recovery, can water purification be dispensed with. Even then it would seem wise practice to use chlorination as an additional safeguard.

The standard of bacterial quality of water as delivered to the consumer, which has been most widely accepted, is that adopted by the U. S. Treasury Department for drinking water provided by interstate carriers. This standard as revised in 1925 is essentially not more than 1 *B. coli* per 100 cc. of water. The effect of industrial wastes on public water supplies is considered in Chapter XX.

287. Protection of Bathing Beaches.—Generally speaking, it would be ideal for the water of bathing beaches to approach drinking-water quality. The standard of bacterial quality of swimming pool waters recommended by the Joint Committee on Bathing Places of the American Public Health Association and the Conference of State Sanitary Engineers suggests a *B. coli* limit of 3 to 4 per 100 cc. It is doubtful if this limit can be maintained for bathing beaches. Disinfection of the waters in the vicinity of bathing beaches by means of chlorine apparatus on boats is practiced at a number of places.

288. Conservation of Useful Aquatic Life.—Most food fish are sensitive to pollution, largely because they require the presence of a moderate quantity of dissolved oxygen in the water.

This has been emphasized in disputes over the permissible pollution of New York Bay, where the dissolved oxygen should not be reduced below 70 per cent saturation, according to Black

and Phelps, in order that food fishes might continue to live in the waters. Fuller stated[1] on this topic:

> With respect to guarding against objectionable odors I think it is clearly necessary for the chemists and bacteriologists to keep in mind that putrefaction does not exist so long as oxygen remains at all. In fact you can go further than that, and say that so long as oxygen is available from nitrates, nitrites, or other oxidized salts there is substantially no putrefaction. I am aware that that does not provide for one feature that may be of some importance, and that is the protection of major fish life. I believe, however, that the European custom in many places is sound in indicating that 30 per cent of the dissolved oxygen necessary for saturation provides a reasonable margin in the case of a majority of species of fish life of the larger kinds. Perhaps some may call for more, but so long as there is 30 per cent remaining at all places at all times, it is a matter of deduction from our well-established laws of biology and chemistry that there can be no putrefaction. The larger number of the principal rivers in this country serving as public water supplies do not contain as much as 70 per cent of the oxygen necessary for saturation. Among the rivers with which I have been personally familiar through analysis, I may mention the Merrimack River at Lawrence, Mass. Twenty years ago it had as low as 50 per cent and sometimes but 30 per cent of dissolved oxygen. In those days it served as the water supply for Lawrence without being filtered, and in the last 17 years, since filtering, it has been regarded as one of the good water supplies of the world. I believe that if this 70 per cent margin suggested by Dr. Soper were applied to Lawrence, it would show that the Merrimack River at that place was not providing a proper disposal for the sewage at Lowell and the cities above, notwithstanding the fact that it provides an excellent water supply at that point.

A somewhat different problem is that connected with the pollution of shellfish beds. Unlike fish, oysters and clams are often consumed raw and may therefore become vehicles of infection by harboring in their bodies pathogenic organisms ingested from the waters in which they abound. A number of epidemics of typhoid fever have been traced to contaminated shellfish.

From the idealistic standpoint, shellfish beds should be located in waters which meet the bacterial standards for drinking water. Experience indicates, however, that such a standard is unnecessarily rigorous. In Massachusetts (1929) the rating of shellfish areas is based upon the results of the sanitary survey, the *B. coli* content of the waters at the areas in question and the *B. coli* content of the shellfish.

[1] *Trans.*, Am. Inst. Chem. Engrs., **3**, 392.

If oysters are relaid in clean water they will quickly purge themselves of pollution. This offers a means, as yet insufficiently explored, for utilizing oysters which originate in undesirable areas. Chlorination of oysters and clams after removal from their beds also seems to render them safe for consumption. The practice of *bloating*, *freshening* or *bleaching* oysters by placing them in brackish water for a day or two appears to be regarded with disfavor. Too often the bloating is accomplished in coves, bays or the mouths of streams which, being closer to shore than the oyster beds, are apt to be more heavily polluted. The bloating process should not be confused with the fattening process of transplanting oysters to clean beds in shallow water rich in food stuffs, chiefly algae (diatoms).

DILUTION PROJECTS

289. Types of Dilution Projects.—Several types of dilution projects may be distinguished according to the nature of the water courses into which sewage is to be turned, namely (1) inland river systems; (2) large fresh-water lakes; (3) tidal estuaries and the ocean. A fourth type of dilution project is one like the Chicago Drainage Canal project, in which a water course otherwise not adequate for dilution is rendered so by engineering means, such as canal construction, pumping, or both. All four types of projects commonly involve studies of (*a*) hydrographic conditions, including the volume and nature of the diluting water; (*b*) sanitary conditions, including the chances for nuisance and direct injury to the public health; (*c*) treatment methods available for overcoming the limitations of the dilution process. The various factors which may play a part in dilution projects have been discussed in this chapter. There remains the illustration of the outfall structures employed and the methods of securing adequate dilution.

A fifth type of project is the discharge of settled sewage into ponds in which fish are raised. This process has been employed as yet only in Europe.

The relative use in the United States of river, lake and ocean outfalls and land disposal systems is shown for the year 1928 in the summary at the top of the next page, of the disposal methods of the 68 cities that had a population of more than 100,000 in 1920. The values given are expressed in per cent of the total population of about 27.5 millions in the cities included.

Artificial dilution is at present resorted to in Chicago and Brooklyn.

Tributary to	Percentage of total population	
	Including as river outfalls all those discharging into rivers, even though close to lake or ocean	Including as lake or ocean outfalls, those which discharge into rivers at short distances from lake or ocean
River outfalls	55.9	35.3
Lake outfalls	7.6	13.1
Ocean outfalls	35.9	51.0
Irrigation works	0.6	0.6

290. River and Lake Outfalls.—If sewage is discharged into a stream flowing rapidly at all times, the outlet need not be submerged, provided the sewage passes into the stream at a point where it is certain to be carried away quickly and at the same time thoroughly mixed with the river water. By a study of the variations in river stage it is possible to locate the outlet so that the sewage will always be discharged directly into water and will not pass over the river bottom exposed by low water. The way of assuring this in the case of the combined sewer outlet at Minneapolis, Minn., was shown in Fig. 83.

Cities situated on the Great Lakes and other large inland lakes are commonly dependent upon these waters both for their water supply and for the disposal of their sewage. The location of outfalls must, therefore, be investigated with particular care in relation to water works intakes. Bathing beaches and shore fronts, too, must be protected against pollution. In order to disperse the sewage thoroughly through a large volume of water before it can rise to the surface, lake outfalls are usually submerged to a considerable depth and are often carried long distances from the shore. At Rochester, N. Y., for example, a 66-in. steel outfall sewer, laid in Lake Ontario, terminates in a timber crib, 7,000 ft. from shore in 50 ft. of water. The crib is 46 ft. square and 24 ft. high. It is weighted with stone and surrounded with riprap extending 10 ft. up the sides. The outlet is 10 ft. above the bottom of the lake.

Whipple's estimates of the path of pollution with on- and off-shore winds during the summer, when the pollution of nearby bathing beaches is of importance, are shown in Figs. 139 and 140.

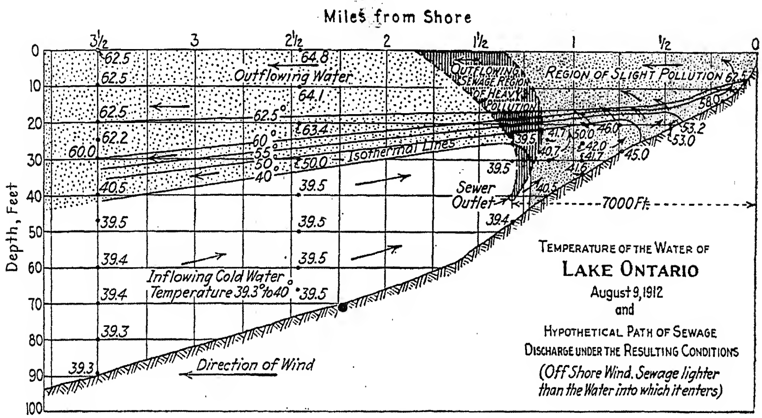


FIG. 139.

It was assumed in these estimates that the sewage is warmer than the bottom water, but colder than the surface water. The student should make his own estimate on the basis that the latter

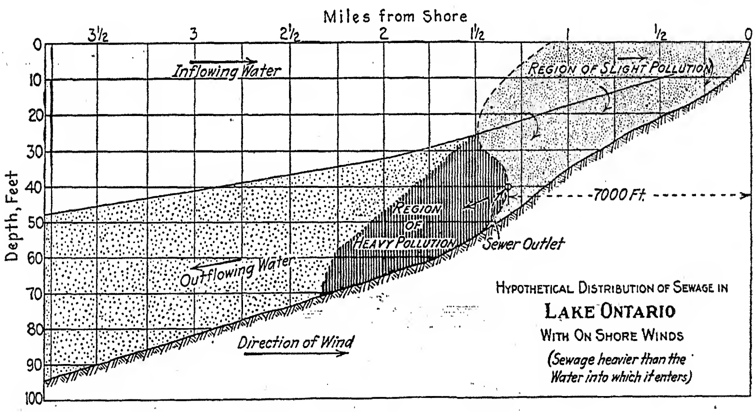
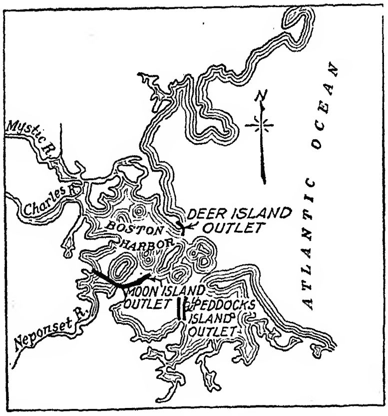


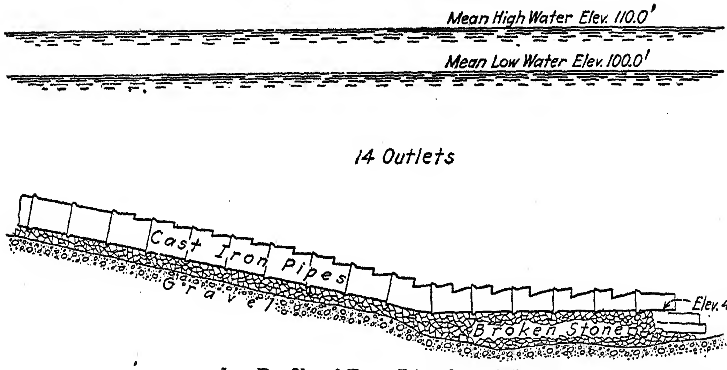
FIG. 140.

assumption is not correct. The sewage discharged through the Rochester outlet is settled in Imhoff tanks prior to disposal.

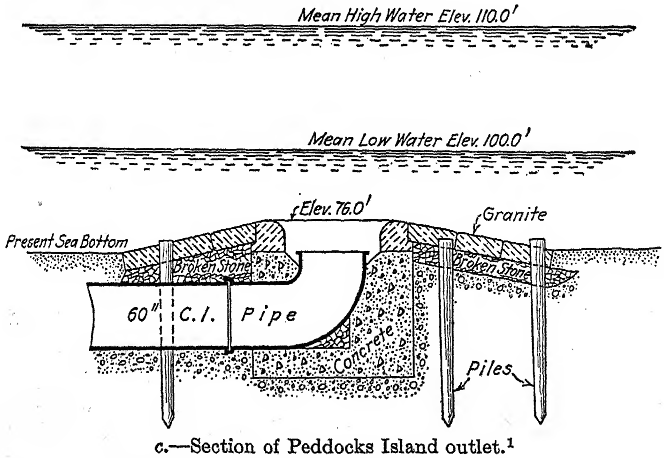
291. Ocean Outfalls.—As in the case of lake outfalls, experience with the discharge of sewage in tidal waters has shown that the



a.—Map showing locations of outfalls.



b.—Profile of Deer Island outfall.



c.—Section of Peddocks Island outlet.[1]

Fig. 141.—Outfalls of Metropolitan Boston.

[1] Note that difference in elevation between outlet and low water is not shown to scale.

outlet should be at a considerable depth, in order to disperse the sewage through as large a volume of water as possible before it rises to the surface. Not only do the large solids in sewage present an offensive appearance if they float on the surface, but the greasy substances cover the surface with a film or "slick" which, although not necessarily an indication of putrescible organic matter, is yet unsightly and likely to be the cause of complaint against the disposal of the sewage in this way. These undesirable conditions are much reduced when the sewage is discharged in depths of 25 ft. or more. The development which has taken place in the design of ocean outfalls is illustrated by the three disposal projects of Metropolitan Boston, shown in Fig. 141.

The first large outfall sewer which was built to discharge into Boston Harbor has its outlet near Moon Island, where float observations have shown that the average velocity of the ebb tide is 0.74 mile per hour and that the floats travel about 4 miles seaward during the whole ebb tide. The sewage is stored in tanks on the island and discharged during ebb tide. Experiments show that when 22 million gal. of sewage are discharged in 45 min., about 750 acres of harbor surface are plainly discolored on a comparatively calm day, and an objectionable appearance is presented by two-thirds of this area, although offensive odors arise from but a relatively small portion of it. Suspended matter is sometimes seen 1½ miles from the outlet, and slick is occasionally observed at still greater distances. Generally speaking, the upper 2 to 3 in. of the sewage-covered area contain much the greater percentage of the sewage. The conditions described last for 2 to 3 hours, depending largely on action of waves.

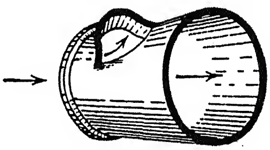


FIG. 142.—Typical outlet pipe, Deer Island outfall.

The second large outfall sewer discharging into the harbor had its outlet originally near Deer Island Light, but a few feet below the surface and near the edge of the main ship channel. The sewage was discharged continuously and discolored the water over an area of 350 acres on ebb tide and for a distance of 1½ miles from the outlet, although slick was sometimes observed farther out on calm days. In 1916 this outfall was extended 315 ft. farther out toward the ship channel, discharging through 14 openings, the deepest being at a considerably greater depth than the discharge end of the original outfall. Of the multiple openings the outermost is 48 in. in diameter and the others are elliptical, varying from 25 by 44 to 13 by 23 in. A typical outlet pipe is shown in Fig. 142.

The third large outfall sewer has its outlet near Peddocks Island at a depth of 24 ft. at low tide, where the sewage is discharged continuously. The sewage is diluted so quickly by the sea water that the percentage of sewage in the surface water directly above the outlet is relatively small.

The use of multiple outlets has found its greatest development in the outfall of the Passaic Valley Sewerage System in New York Harbor. The arrangement of this outfall, together with the special nozzle used to insure thorough mixing of the sewage with the harbor water, is shown in Fig. 143.

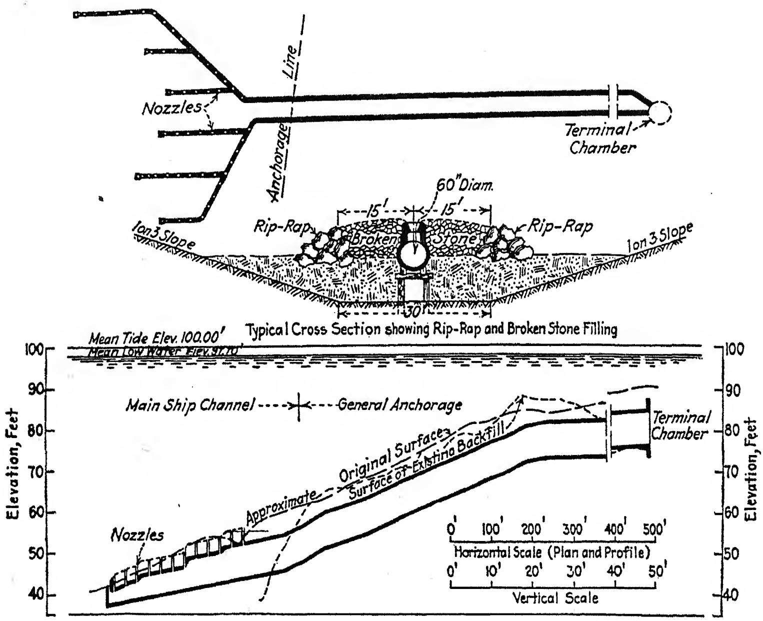


FIG. 143.—Passaic Valley outfall in New York Harbor.

292. Artificial Dilution.—Until the intercepters along the three rivers passing through the City of Milwaukee were completed, much of the sewage of the city was discharged directly into these water courses. With the rapid growth of Milwaukee the streams became grossly polluted. Water was therefore pumped from the lake, as shown in Fig. 144, carried through what are called *flushing tunnels,* in reality water conduits, and discharged into the streams to swell their volume during times of low flow and warm weather. This is an example of artificial dilution. Flushing works of this kind were devised too for Chicago and Brooklyn. At Milwaukee the construction of intercepters and treatment works was completed a few years ago.

Chicago offers an example of a different type. To protect the waters of Lake Michigan a canal was built connecting the Chicago

River with the Des Plaines River, which flows into the Illinois and then the Mississippi River (see Fig. 145). The flow of the Chicago River was thus reversed and water abstracted from Lake Michigan serves to dilute the sewage of the city which is discharged into the canal. To reduce the amount of diluting water required, as a result of the rapid growth of the district and the enormous development of its industries, treatment works fed by

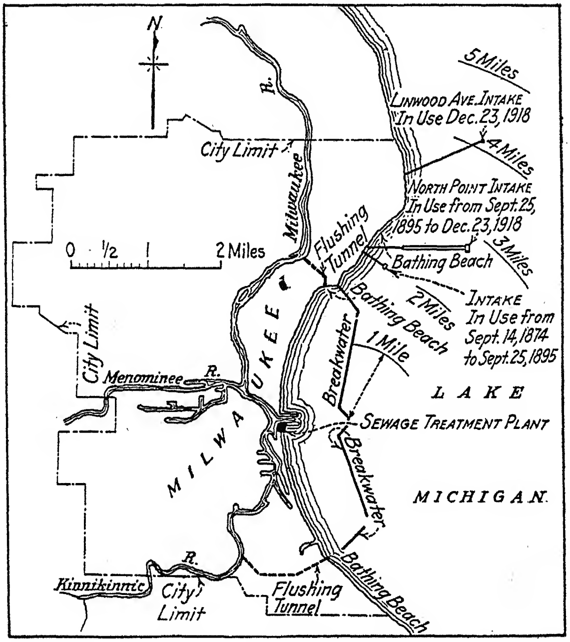


FIG. 144.—Map of Milwaukee showing arrangement of flushing sewers.

intercepting sewers and discharging into the canal are being put into operation.

293. Fish Ponds.—A method of sewage disposal involving dilution is the discharge of settled sewage into shallow artificial ponds which are stocked with fish. Reclamation of the nutritive elements contained in sewage, through the intermediate activity of bacteria, plankton and other small aquatic organisms, in the form of fish flesh was first emphasized by B. Hofer, Professor of Zoölogy in the University of Munich. The feasibility of this process must be apparent from what has been said in this chapter about the succession of life in sewage-polluted streams. The

difficulties encountered relate to (1) the maintaining of a sufficiently high oxygen concentration by keeping the sewage fresh, avoiding sludge deposits and destroying surface growths which prevent reaeration or absorb oxygen from the water; (2) the elimination of toxic substances, such as hydrogen sulphide, and of flocculent masses of iron hydrate and the like which deposit on the gills of the fish and cause asphyxiation, and (3) the main-

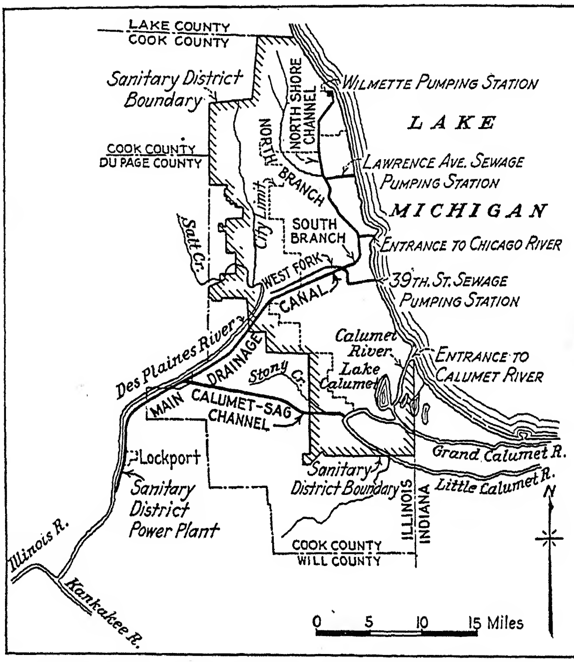


FIG. 145.—Location of rivers and canals; Chicago, Ill.

tenance of a biological balance that will yield adequate quantities of fish food.

The process has as yet not been applied in the United States. As practiced in Central Europe, dilutions of 2 to 5 volumes of clean water are employed for settled sewage. The ponds are 1 to 2½ ft. deep, one acre of pond surface being provided for 800 to 1,200 persons. The ponds are drained and cleaned during the winter when ice conditions and retardation of life processes interfere with the raising of fish. The sewage is disposed of by other means during this time. Special hibernating basins are

provided at Strasbourg for the plant and animal pond-life. In the spring the ponds are filled and stocked with fish. Ducks keep the ponds clear of undesirable weeds.

The fish raised are usually carp and tench, both highly prized in Europe, but not in America. When the dilution is great, rainbow trout also are stocked. A pond of one acre area is said to produce from 400 to 500 lb. of fish flesh and from 200 to 250 lb. of duck meat. The most notable plant in Europe is the one at Strasbourg, where the fish ponds remove 88 per cent of the organic material and 80 per cent of the nitrogen from the sewage. The effluent is clear and contains about 10,000 bacteria per cubic centimeter. A large plant is in process of development at Munich, Germany.

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Problems

1. With the aid of Table 73, prepare a diagram showing in tiers the succession of life and the changes in oxygen and carbon dioxide in the Genesee River at Rochester. Plot the bacterial counts on a logarithmic scale, the remaining information on an arithmetic scale.

2. The sewage from a manufacturing city of 100,000 people is discharged at the rate of 100 gallons per capita daily untreated into a stream with a dry-weather flow of 3.5 c.f.s. per 1,000 population. Assuming the following conditions: 5-day, 20°C., B.O.D. of sewage, 250 p.p.m.; temperature, 75°F.; oxygen saturation, 50 per cent; 5-day, 20°C., B.O.D. of river water, 20 p.p.m.; temperature, 70°F.; oxygen saturation, 70 per cent; deoxygenation constant at 20°C., 0.1; reaeration constant at 20°C., 0.2; draw the 10-day dissolved oxygen sag.

3. If in Problem 2 the velocity of the stream is ⅔ mile per hour, how many miles will elapse before the river water is 40 per cent saturated with oxygen—(*a*) during degradation, (*b*) during recovery? *Ans.* (*a*) 4.8 miles.
(*b*) 134 miles.

4. Assuming that the first-stage, 20°C., B.O.D. of a sewage is 288 p.p.m., that it contains 5 p.p.m. of oxygen, and that one volume of the sewage is diluted with four volumes of river water saturated with oxygen, how soon will the oxygen be exhausted if the average temperature of the stream is 4°C. and the stream is covered with ice? *Ans.* 3.1 days.

5. A tidal estuary is found to contain 1,250 p.p.m. of chlorides. The river emptying into it contains 10 p.p.m. of chlorides. What is the per cent of river water in the estuary? *Ans.* 93.1 per cent.

6. Find the dilution afforded a sewage flow of 5 m.g.d., which is discharged at the upper end of a tidal estuary with a surface area of 600 acres at mean low water and 640 acres at mean high water, the mean range of tide being 8 ft. Two and one-half successive ebb tides are required to carry the sewage seaward from the outfall to a point from which it will not return to the estuary, and the average time for one round of tides is 12 hours 23 min.
Ans. 1:250.

7. Plot the values for bacterial death rates shown in Table 74, on semi-logarithmic paper.

8. (*a*) What is the 5-day, 20°C., B.O.D. corresponding to the General Standard of the Royal Commission? (*b*) What is the first-stage B.O.D. at 20°?

Ans. (*a*) $\geqq$ 21.7 p.p.m.
(*b*) $\geqq$ 31.9 p.p.m.

9. What is the dilution ratio corresponding to a flow of 4 c.f.s. per 1,000 inhabitants in an average American city? *Ans.* 1:25.8

10. Using the U. S. Public Health Service observations at Cincinnati, how many days must elapse along the Ohio River below the sewers of a city before the raw water count will be such that the chlorinated effluent of a rapid sand filter will constantly show less than 0.5 *B. coli* per 100 cc.? Assume that the dilution is 1 in 10,000 and that the sewage contains on an average 500,000 *B. coli* per cubic centimeter. *Ans.* 1.25 days.

11. Assuming that the water intake shown in Fig. 135 is a sewage outfall discharging sewage with a temperature of (*a*) 73°F., (*b*) 64°F., make a sketch showing the probable path of the sewage with on- and off-shore winds assuming that the temperatures of the lake water are as shown.

12. A cast-iron outfall discharging in 40 ft. of sea water is to carry 1 m.g.d. of sewage from a separate system to a distance of 5,000 ft. from a pump on shore. Assuming a fluctuation in flow of ± 25 per cent, (*a*) what must be the diameter of the pipe and (*b*) what must be the total lift of the pump if the pump is 10 ft. below sea level? Deposition of solids in the outfall is to be prevented. *Ans.* (*a*) 10 in.
(*b*) 49.5 ft.

CHAPTER XIII

SEWAGE DISPOSAL BY IRRIGATION

294. Definitions.—Besides sewage disposal into water, called *dilution*, there is sewage disposal upon land, called *irrigation*. Both are essentially disposal processes as opposed to treatment processes, because uncontrolled natural purifying agencies are relied upon to render the sewage matters inoffensive. Dilution makes use of the forces of purification operative in water; irrigation calls upon those active in soil. If irrigation also is classified as a treatment process, this is due chiefly to the fact that irrigation ditches and sometimes underdrains must be provided and that many sewage irrigation plants have effluents which are discharged into streams or other bodies of water for final disposal.

Two types of irrigation projects are distinguished—*sewage farming* and *subsurface irrigation*. Sewage farming, also known as *broad irrigation* and *land treatment*, dates back to the earliest days of sewerage systems. In this process sewage is caused to flow over cultivated fields or to percolate through the ground until it joins the natural ground water or passes into underdrains, incidentally watering and to some extent fertilizing the growing crops. In *subsurface irrigation* sewage is distributed beneath the surface of the ground and penetrates into the soil from open-jointed pipes. While sewage farming has been employed for very large communities, as well as for smaller ones, subsurface irrigation is confined to small water-carriage systems, more particularly to those of isolated dwellings, hotels, country clubs and institutions.

Irrigation with sewage in the form of sewage farming is carried out with two objects in view; (1) proper disposal of the sewage, and (2) cultivation of crops from which revenue may be obtained. The disposal of sewage should be the primary object, in most cases, and the raising of crops secondary, controlled so as not to interfere with the purification of the sewage. In subsurface irrigation the cultivation of crops is only considered incidentally.

295. Purification Processes Operative in Soil.—As in dilution, the processes of purification operative in soil are physical, chemical and biological in nature. They are akin to the forces of self-purification encountered in water, but are more restricted in range of activity.

Physical Processes.—The physical processes of purification are greatly dependent upon the character of the soil. Two of them are particularly important, namely *filtration* and *aeration.* Passage of sewage through the soil removes larger solid particles by straining, and smaller ones by sedimentation in the quiescent portions of the tortuous channels between the soil grains or by adsorption on the many contact surfaces. The quantity of sewage that can percolate through the ground is obviously dependent upon soil texture. A stiff clay will allow little water to pass, while a coarse sand will absorb a great quantity. For a discussion of the principles of filtration and for their formulation, see Chapters XIV and XVII.

The process of aeration which is active in irrigation is much the same as that taking place in sewage disposal by dilution. On the land surface oxygen is absorbed from the free atmosphere; within the soil, oxygen is taken up from the ground air. The sewage as it recedes from the surface draws in behind it a fresh supply of air, which serves to maintain aerobic conditions in the soil. This *breathing* of the soil is very important in the economy of irrigation projects, just as reaeration is in dilution. In order to permit a replenishing of the oxygen supply, sewage must be applied to land intermittently. A rest period must be provided. It stands to reason that aeration is most active in the upper strata of the soil.

An open sandy soil would appear to be the most favorable type for sewage treatment by irrigation, because of its greater permeability to both water and air. In sewage farming, however, no matter what the character of the underlying strata, the loam and vegetation will hinder the passage of water and the circulation of air so that sewage farms can successfully treat but a small percentage of the volume that can be handled on uncultivated sand areas such as are used in the treatment of sewage by what is called *intermittent filtration* (see Chapter XVII).

Chemical Processes.—Of the chemical processes which are active in irrigation, oxidation is the most important. Aerobic decomposition carries the cycle of organic matter to a point where the end products are not only inoffensive but where they become

available as plant foods. This fertilizing value of sewage is discussed in Section 296.

Biological Processes.—Bacteria abound in moist soil, where they are responsible for the rotation of the elements which enter into the structure of organic matter, just as they are in water. Other organisms, too, are found in sewage fields, but life as a whole is less varied than in sewage polluted water courses. As sewage matters become converted by biochemical processes into substances which will serve as plant foods, they are taken up by the roots of the irrigated crops.

296. Action of Sewage on Crops.—Sewage contains chemical constituents which, when present in suitable form, possess fertilizing value. For this reason there has been a popular belief that the agricultural utilization of sewage will be profitable. The most important plant food is water, which is the main constituent of sewage (99.9 per cent). Probably the chief agricultural value derived from sewage when used for irrigation is due in most cases to its water content. Only portions of the nitrogen, phosphorus and potassium compounds in the sewage are found in a form suitable for direct utilization by plants. Nitrogen should be present as nitrate to be of most value. Usually only a very small part of it, or none at all, has arrived at that state of oxidation when the sewage is applied to land. The remainder must be nitrified by biological agencies which require oxygen, a mild temperature and the presence of lime or some other base. The phosphates and potash, especially the latter, are present in such small quantities that their fertilizing value is usually small compared with that of the nitrogen compounds that can be converted into available form.

The fertilizing constituents of sewage are associated with fat and soap, which are injurious to land. They clog the pores of the soil and interfere with the absorption of the sewage by the soil and the subsequent aeration of the pores. Land which has become clogged and rank in this way is called *sewage sick* and one of the most important duties of the manager of a sewage farm is to prevent such a condition from becoming chronic in any of his fields. A precautionary measure against it is the removal of the coarse suspended matter from the raw sewage by screening and sedimentation.

In agriculture, fertilizers should be applied at certain stages in the rotation and growth of crops, and the proper fertilizers to

use depend upon the nature of the soil, the climate, the crops to be grown and the rotation of crops. In sewage disposal, all these considerations must be waived in favor of the production of a satisfactory effluent. The crops must be regarded merely as by-products. Evidence furnished by long experience in a number of countries under many conditions does not reveal that it is practicable to obtain much fertilizing effect from city sewage by the means that must be used to obtain successful treatment, but indicates that where irrigation has been successful agriculturally the same results would have been produced with water.

A great many varieties of crops have been grown on sewage farms. The general classes of vegetation cultivated are: grasses; coarse beets and other fodder; kitchen vegetables, especially cabbages; corn and wheat; and groves of walnut and orange trees.

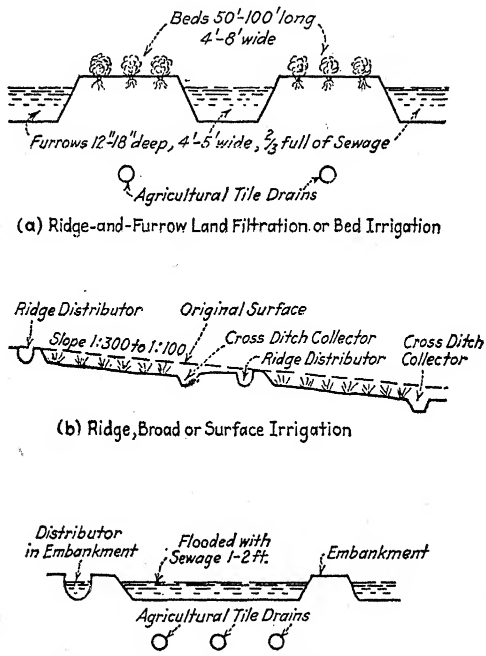
297. Construction of Irrigation Areas.—Sewage is applied to land in various ways, some of which are illustrated in Fig. 146. If it is flowed over sloping fields from an upper level to a lower one, the process is called *ridge, surface* or *broad irrigation.* In this method the sewage does not penetrate more than a few inches into the soil, except where the latter is unusually porous. When the sewage is applied by any method to soil of such a nature that the sewage percolates downward in considerable quantities, the process is called *filtration.* This variety of filtration must be carefully distinguished from *intermittent sand filtration,* which is practiced on specially prepared areas that are not used for agricultural purposes, wherever this method of treatment is carried on most efficiently.

Ridge, surface or broad irrigation (Fig. 146*b*) is practiced by flooding sewage over land from channels on ridges between long, gentle slopes; the sewage collecting in the low places between the ridges is received by cross ditches at the foot of the field and conducted to ridges in a field lying somewhat lower than the first. Where the soil is very heavy a third field may be laid out.

Bed irrigation and *land filtration* are terms used in Europe to designate the distribution of the sewage in numerous ditches cutting up the land into beds, which are kept moist by horizontal seepage from the ditches (Fig. 146*a*). This method is called *ridge-and-furrow irrigation* in England, and is employed when it is desired to keep the sewage from contact with the crops growing on the beds.

In *flood irrigation* a plot of land surrounded by a bank is periodically covered with sewage to a depth of 1 or 2 ft. (Fig. 146*c*). In Germany, a method of spraying sewage upon the land from movable pipes attached to the sewage conduits has been employed.

To avoid pools of sewage dotted over the sewage fields, the grading of the surface and of the ditches and channels must be carried out correctly. The main carriers are usually constructed



(a) Ridge-and-Furrow Land Filtration or Bed Irrigation

(b) Ridge, Broad or Surface Irrigation

(c) Flood Irrigation or Downward Filtration

FIG. 146.—Construction of irrigation areas.

of masonry or concrete, but for the minor distributing system earth channels are preferred, because they do not interfere with the cultivation of the land which is considered necessary even where no crops are grown, in order to promote aeration and to check weeds. Much more care is paid in England to grading the land for filtration than for surface irrigation, although it is recognized that even with surface irrigation, pooling of the sewage in detached places is likely to result in sewage sickness of the land at such points and in a poorer effluent. The cost of

preparing the land for filtration is much greater than for surface irrigation, not only on account of having the surface perfectly level, but also because it is frequently desirable to strip off the fine surface soil overlying a coarser subsoil.

The underdrainage of land receiving sewage is regarded as having a dual object, (1) the aeration of the soil and (2) the removal of the effluent after it has percolated through the soil. Drainage is frequently carried out by deep ditches into which lines of 3- or 4-in. agricultural drain-tile discharge. It is held by some engineers that open ditches should be used to permit inspection of the effluent and that they should be deep enough to keep the level of the ground water below the level of the tile drains, so that the soil may be kept well aerated. Instead of open ditches, tile pipe drains often are employed. The general opinion is that all land except clay needs underdrainage when the ground-water lies within 4 ft. of the surface. Clay is difficult to underdrain because the tile seems to increase the number of cracks which open in dry, hot weather and permit raw sewage to enter the drains in an unchanged condition. The underdrains of English sewage farms, according to Kershaw,[1] vary from 2 to 6 in. in size, from 12 to 66 ft. in spacing, and from 3 to 9 ft. in depth. The main drains are 4 to 24 in. in diameter.

The area required for sewage farming depends upon the nature of the soil and the method of irrigation as well as upon the skill of operation. It varies too with the degree of preliminary treatment which the sewage has received. Common values of the amount of sewage which can be treated on an acre of land vary from 2,000 to 40,000 gal. daily, with an average of 8,000 gal. Under European conditions these values correspond to a population of 50 to 1,000 with an average of 200 persons per acre. Since irrigation is controlled in large measure by the water load, the population figures must be reduced to about 60 per cent, to compare them with American conditions. From two to four times as much land is required for surface irrigation as for filtration.

Irrigation areas must rest about half of the time. Dosing periods vary from 12 hours to 10 days at different farms. In addition to the short periods between doses, there must be provided longer *rest* or *drying-off periods,* to permit the land to recover and to allow for the harvesting of crops. In England from

[1] "Modern Methods of Sewage Purification."

20 to 50 per cent of the area is under irrigation at one time. Longer resting cycles are required for surface irrigation than for filtration.

Storm waters to the extent of three times the dry-weather flow are handled by European sewage farms. Flows in excess of this rate commonly are bypassed after flowing through storm-water basins. During freezing weather it is often necessary to store the sewage in detention basins, which commonly take the form of open earthen basins.

Compared with the rainfall the quantities of sewage used for irrigation are large. At Nottingham, England, where filtration is practiced, for example, the annual rainfall is 25 in., while the annual depth of sewage is about 18 ft. At South Norwood, England, where surface irrigation is employed, the values are 24 in. and 5 ft. respectively.

298. Operation of Irrigation Areas.—The Royal Commission on Sewage Disposal has given detailed attention to the operation of sewage farms in the British Isles. Eight farms typical of different conditions were kept under close observation for two years, and observations were made less regularly at other farms.

At these eight farms from 12 to 53 per cent of the area was not irrigable, either because it was unsuited for the purpose or had not been prepared to receive sewage, or was required for roads, buildings, sludge beds and other purposes.

The Royal Commission's conclusions from its investigations were stated substantially as follows in its fifth report (1908):

1. There is no substantial distinction between effluents from land and effluents from artificially constructed filters. Effluents from those soils which are particularly well adapted for the purification of sewage contain only a very small quantity of unoxidized organic matter, and are usually of a higher class than effluents from artificial filters, as constructed and used at the date of the report. Effluents from soils not well adapted to purify sewage may often be very impure.

2. Generally speaking the evidence points to a maximum rate of 36,000 U. S. gal. per acre, or 1,000 persons per acre, with the best land, after preliminary treatment of the sewage, although some witnesses who testified before the Commission put the rate as high as 72,000 gal., or 2,000 persons, per acre, under similar conditions. With unsuitable land, such as clay, not more than 3,600 gal. per acre can be efficiently treated, even after settlement of the sewage.

3. The total acreage of a farm must be relatively much greater when the sewage is purified by surface irrigation than when the method of filtration is employed, and a larger percentage of surplus area is also desirable in the

former case. The larger the total irrigable area the greater is likely to be the working cost. On the other hand, with good management, the larger the surplus irrigable area, the better is the purification likely to be and, within certain limits, the prospect of profit.

4. In the case of sewage which is to be treated upon artificial filters, it is generally desirable to settle out as much as possible of the suspended solids before filtration. This is of less moment in the case of sand, but where the soil is heavy the sewage should first be efficiently screened and settled. Porous sandy soil, worked as filtration farms, may be able to treat crude, unsettled sewage without detriment, but, even in those cases, there is the possibility of nuisance arising from the decomposition of sewage solids on the surface of the soil, and such solids may cause damage to crops.

5. It is impossible to lay too much stress on the importance of sewage farms being well managed. Farm managers have a most difficult part to play, and no amount of care and attention will ever enable land of any kind to deal with a volume of sewage in excess of the effective purifying capacity of the soil. As a rule, sewage farms should not be let.

6. English soils and subsoils may be divided into the following three broad classes: Class I, all kinds of good soil and subsoil, such as sandy loam overlying gravel and sand; Class II, heavy soil overlying clay subsoil; Class III, stiff clayey soil overlying dense clay. Since variations exist in practice, both as regards the method of purification employed and the extent of cropping, the first of these three classes may be divided into three subclasses, as follows: (1) filtration with cropping; (2) filtration with little cropping; (3) surface irrigation with cropping. The method of purification assumed to be suitable for the other classes of soil is surface irrigation with cropping.

Table 76.—Average Amounts of Land Required for Treating a Dry-weather Flow of 1,000,000 U. S. Gallons

(Royal Commission on Sewage Disposal. Fifth Report, page 153)

Class of soil	Method of working	Volume of settled sewage which can be treated per acre per 24 hours U. S. gal.	Total area of land required to treat a dry-weather flow of 1,000,000 U. S. gal.[1] acres
I. Good soil and subsoil	Filtration with cropping	14,400	70
I. Good soil and subsoil	Filtration with little cropping	30,000	33
I. Good soil and subsoil	Surface irrigation with cropping	8,400	121
II. Heavy soil on clay	Surface irrigation with cropping	6,000	167
III. Stiff clayey soil on dense clay	Surface irrigation with cropping	3,600	278

[1] These areas are sufficient for the treatment in times of storm of three times the mean dry-weather flow.

With a heavy soil and clay subsoil, by far the greater part of the purification is effected by surface irrigation, although in exceptional circumstances a good deal is also done by filtration.

Having regard to the volume of sewage dealt with and the purification effected at the farms kept under the Commission's observation, and the testimony given by various expert witnesses called by the Commission, it estimated that the several classes of soil and subsoil could deal effectively with the volumes of settled sewage given in Table 76, under the climatic conditions of England.

299. The Sewage Farms of Paris and Berlin.—The utilization of sewage in one way or another has long been practiced in France. The native thrift of the French people led them many years ago to attempt quite extensive use of the wastes collected in cesspools, and Victor Hugo's urgent recommendation[1] for employing the sewage of Paris in irrigating farms about that city was long ago adopted; but the recovery of wealth expected by him has not been experienced.

Paris.—The sewage farms about Paris are the chief French examples of sewage irrigation. In 1923 the volume of the city's sewage, from a population of almost 3,000,000 in the districts connected with the farms, was more than 200 m.g.d., of which 120 m.g.d. were used for irrigation, the remainder being discharged into the River Seine. The area of the farms is given in Table 77.

The farms owned by the city had cost $7,220,000 in 1900, and the annual cost of operating them and distributing the sewage was about $1,000,000. Financial reports of the farming operations are not obtainable, so it is impossible to ascertain the net cost of this method of treatment at Paris. The city's land was leased to tenants at rates ranging from about $5 per acre, where

[1] "Paris casts twenty-five millions of francs annually into the sea; and we assert this without any metaphor . . . Twenty-five millions are the most moderate of the approximate amounts given by the estimates of modern science. Science, after groping for a long time, knows now that the most fertilizing and effective of manures is human manure . . . There is no guano comparable in fertility to the detritus of a capital, and a large city is the strongest of stercoraries. To employ the town in manuring the plain would be certain success; for if gold be dung, on the other hand our dung is gold . . . Do you know what those piles of ordure are, collected at the corners of streets, those carts of mud carried off at night from the streets, the frightful barrels of the night-man, and the fetid streams of subterranean mud which the pavement conceals from you? All this is a flowering field, it is green grass, it is mint and thyme and sage, it is game, it is cattle, it is the satisfied lowing of heavy kine, at night it is perfumed hay, it is gilded wheat, it is bread on your table, it is warm blood in your veins, it is health, it is joy, it is life." ("Les Miserables," Book II, Chap. I, Jean Valjean.) While the scientists of Hugo's acquaintance may have been assured of the financial figures he gives, it is an unfortunate fact that neither science, engineering nor agriculture has yet been able to obtain these millions represented by sewage, because the expense of building works to utilize sewage and the cost of operating the works exceeds the value of the sewage, and because the requirements of public health are paramount to those of sewage utilization.

the tenant agrees to apply sewage to his land only as directed by the city's supervisors, to about $40 per acre where he uses the sewage when and as he desires. The owners of land within the irrigation districts take the sewage as they desire it.

TABLE 77.—SEWAGE FARMS OF PARIS IN 1924

	Area of farms, acres		
	Privately owned	Owned by city	Total
Gennevilliers	1,880	15	1,895
Achères	410	2,965	3,375
Méry-Pierrelaye	3,731	1,236	4,967
Carrières-Triel	2,137	210	2,347
	8,158	4,426	12,584

The sewage is screened and settled. It is distributed through the farms in reinforced concrete conduits 1 to 4 ft. in diameter, lined with sheet steel where the pressures are heavy. These conduits have risers 1 ft. in diameter with outlets for the sewage into open carriers which distribute it over the fields. The land is partially underdrained at a depth of 13 ft., by drains of plain or reinforced concrete pipe. They discharge into open ditches with concrete lining.

Berlin.—The sewage farms of Berlin afford the largest opportunity for this method of treatment yet (1929) undertaken in any country. At the close of 1910 the area of these farms was as given in Table 78, and they were treating an average of 77,000,000 gal. daily, coming from a population estimated at 2,064,000. Of the prepared land 7,994 acres were used for broad irrigation, 12,250 acres for filtration, 502 acres for settling basins, and 2,105 acres for roads and buildings. The rate of filtration was about 3,700 gal. per acre of prepared land per day. The principal crops were rye-grass, turnips, cabbages, potatoes and grain. About one-fourth of the area was used for pasturage and there were about 40 acres of fish ponds which yielded fish worth about $3,200 annually. In 1926 the Berlin sewage farms were treating about 150 m.g.d. of sewage from 4 million people on 27,250 acres.

The cost of the farms to March 31, 1910, was about $17,470,000. The expenses for the year ending on that date were

$1,300,000 for maintenance and $742,000 for interest and loans. The receipts were $1,241,000 and there was an estimated increase of $122,600 in the value of live stock and other property. The net cost of sewage disposal was therefore about $24 per mil. gal. Similar balances have been maintained during recent years (1925).

TABLE 78.—BERLIN SEWAGE FARMS IN 1910

	Farmed by city	Leased to farmers	Unproductive	Total
Area irrigated and farmed, acres..	16,657	3,956	395	21,008
Area farmed without irrigation, acres........................	10,647	2,486	8,868	22,001
	27,304	6,442	9,263	43,009

300. American Sewage Farming.—Sewage irrigation has not been tried in the United States on a large scale, but it has been practiced in a small way at a number of places, particularly in Southern California, for many years. The cities of Danbury, Conn., Pasadena, Fresno and Pomona, Cal., and San Antonio, Tex., are among the places where it has been given long trials with more or less success. Some of these places, as they grew larger, found this method impracticable and have abandoned it for other methods. Health officers report from time to time that sewage is treated by irrigation when what is done is to raise crops on intermittent sand filters, thereby running the risk of clogging the filters by roots and necessitating an early and expensive reconstruction of the beds. At one time irrigation was practiced extensively by market gardeners near Los Angeles, but it was unsatisfactory both to the health officers and to the farmers. Cheyenne, sometimes mentioned as a city using sewage for irrigation, has not done so since 1890. Pullman, Ill., now a part of Chicago, the first large American city to irrigate farming land with sewage, gave up the practice many years ago.

The more extended use of irrigation in Southern California than elsewhere is probably due to the suitable character of the soil for sewage farming, to the low rainfall of 12 in. or less per year, and to the scarcity of water. The first condition materially

reduces the cost of this method of treatment, while the last two greatly facilitate using the sewage in a fairly satisfactory agricultural way which does not have decided sanitary defects.

The sewage of San Antonio, Tex., is conveyed 12 miles from the city to a 6,700-acre privately-owned tract, where it is used in irrigating 4,000 acres. The sewage amounts to about 20 m.g.d., so that the land is dosed at the rate of 5,000 gal. per acre daily. The irrigating season is from the middle of February to the middle of November and during the remaining 3 months of the year the sewage is discharged into an artificial lake having an area of about 1,000 acres.

301. Live Stock on Sewage Farms.—Kershaw[1] states that "cattle should rarely be permitted to roam loose on land used for sewage treatment, on account of the damage they do to land—especially heavy soils—by 'pocketing' it with footprints, in which the sewage or tank liquor after application to the land, stagnates and smells when the field is dried off . . . Where surplus land is available on the farm, or where the cattle are stall-fed, this objection naturally does not arise, and heavy crops of rye grass can be disposed of regularly where cattle are kept."

At Melbourne, Australia,[2] cattle and sheep are bred and fattened and horses are pastured on the municipal sewage farm. Altogether there were grazed and fattened on the 8,084 acres under irrigation (1926) from 6,000 to 8,000 head of cattle and from 5,000 to 17,000 sheep, while from 500 to 1,000 horses were on pasturage. Dairying is not permitted by the health authorities. The revenue from grazing more than covers the expenses of land treatment apart from interest on capital. During drought years the revenue also covers interest. The population of Melbourne is about 800,000 and the annual rainfall averages 18 in.

302. Efficiency of Land Treatment.—The efficiency of sewage farms as measured by chemical and bacteriological tests varies with the soil, with the construction and the operation of the farms, and with the season of the year. Analyses of the sewages applied to the farms at Nottingham and South Norwood, England, and at Paris and Berlin are given in Table 79. At the Nottingham farm, which is still in operation, the soil is a light loam whereas at the South Norwood farm it was heavy loam and

[1] "Sewage Purification and Disposal," Cambridge University Press, 1915.

[2] *The Engineer*, 1926; **142**, 519.

TABLE 79.—ANALYSES OF APPLIED SEWAGE AND EFFLUENTS OF EUROPEAN SEWAGE FARMS

(Results of chemical analyses in p.p.m.)

	Nottingham[1]		South Norwood[1]		Paris[2]		Berlin[3]	
	Raw	Effluent	Raw	Effluent	Raw	Effluent	Raw	Effluent
Nitrogen								
Total	76.9	22.7	52.0	23.0				
Organic	31.5	.....	14.9	....	6.7	0.0		
Albuminoid	14.5	0.3	6.7	1.0	.....	....	99.5	2.3
Ammonia	39.8	1.3	35.4	8.7	15.4	0.0–0.2		
Nitrite	.....	20.6		3.9	.....	....	.....	146.6
Nitrate	.....		.....		0.8	19.1–21.0	.....	
Oxygen consumed	232.1[4]	1.9[4]	77.1[4]	14.4[4]	.....	....	333.7	33.6
Dissolved oxygen	.....	7	.....	3.7				
Carbon dioxide	.....	120	.....	55				
Solids								
Total	.....	.....	.....	.....	.....	....	978.4	987
Mineral	.....	.....	.....	.....	152	18.2–30.7	693.2	863
Organic	.....	.....	.....	.....	33.4	0.9–1.7	285.2	124
Suspended	519	.....	219					
Bacteria per cc.								
Gelatin 20°C	23,350,000	21 to 1,540[5]	48,600,000	6,700 to 1,620,000	148,300,000	125 to 1,000		
Agar 37°C	5,150,000	3 to 547,000[5]	6,360,000	980 to 83,000				
B. coli	> 100,000	0 to 500[5]	> 100,000	10 to 5,000				

[1] Royal Commission on Sewage Disposal, 4th and 5th Reports, 1904–1908.
[2] Bechman, 1901. For results for the year 1922 see DIÉNERT, *Rev. d'Hygiene*, 1924; **46,** 1117.
[3] Dunbar, 1908.
[4] Four hours at 80°F.
[5] With one exception.

clay. These differences in soil permitted the use of filtration at Nottingham, whereas surface irrigation had to be resorted to at South Norwood. At Berlin the soil is a light-brown sand with an effective size[1] of 0.13 mm. and a uniformity coefficient of 2.5. The Nottingham sewage is screened while the other sewages are screened and settled. The greater efficiency of filtration as compared with surface irrigation is apparent from a comparison of the Nottingham and South Norwood results shown in Table 79. This difference is readily understandable when it is remembered that in surface irrigation the sewage is merely strained through the crops on the surface of the soil, whereas in filtration the sewage passes downward through the soil.

303. Public Health Aspects of Sewage Farming.—From the hygienic standpoint there are two phases of sewage farming that must be considered, (1) its ability to produce a satisfactory effluent, and (2) the possible dangers from the sewage as it passes over or into the soil. In regard to the first point Table 79 shows that the purification effected by land treatment is relatively great, and comparison with the so-called artificial treatment methods discussed in the succeeding chapters will indicate that the effluent from well-managed sewage farms is as good as that produced by most of the treatment processes that make use of sedimentation followed by oxidation. This is true more particularly of filtration than of surface irrigation. Pollution of water courses, therefore, may be greatly reduced by land treatment of the sewage prior to its discharge into them.

As far as the second point is concerned the Royal Commission on Sewage Disposal expressed its opinion as follows in its fourth report:

> As regards the likelihood of sewage farms being dangerous to health, we can do no more than tentatively express the opinion that no convincing proof has yet been furnished of *direct* or wide-spread injury to health in the case of well-managed farms.
>
> It may be possible that the foul emanations from a badly-managed or over-sewaged farm constitute an indirect source of danger to health by lowering the vitality of weakly or susceptible individuals.

Possible channels of infection are flies and other insects carrying germs of disease mechanically from farms to human habitations; kitchen vegetables consumed without cooking; the milk of cows grazed on irrigated lands; and ground water polluted by sewage

[1] See Chapter XVII.

seeping into it from the filtration areas. In the absence of epidemiological experience, considerations of hygiene dictate that a safe policy be adopted. Dairying and the raising of produce to be consumed raw, even though sewage does not come into direct contact with it,—as is true of some vegetables grown on ridge-and-furrow areas and of the fruits of some sewage-irrigated trees,—should not be allowed on sewage farms. Ground water supplies in the vicinity of sewage areas should be closely watched for evidences of contamination.

In general the average layman has a strong prejudice against the use of foodstuffs grown on sewage farms, and the odors arising from poorly-managed irrigation areas are apt to strengthen his prejudice in spite of the lack of scientific support. It is said that the 4,000 people resident on the Berlin sewage farms have consistently enjoyed good health.

304. Present Status of Sewage Farming.—The fact that sewage farming is the oldest method of sewage treatment, if it be regarded as a treatment process, should not detract from its value. This view is supported by the fact that the municipality of Berlin is abandoning more recent and thoroughly efficient trickling filter installations for sewage farms. With other methods of treatment available, however, sewage farming can compete with the so-called artificial treatment methods only when (1) wide tracts of suitable land are available at low cost; (2) efficient management both from a sanitary and agricultural viewpoint is assured; (3) water is scarce and hence valuable.

In America, therefore, sewage farming seems practically restricted to the semi-arid regions of the Southwest. In England its use is decreasing rapidly. In France other processes are being studied with a view to abandoning some of the existing farms. In Australia, South Africa and countries with similar climatic and soil conditions, however, extensions to sewage farms are being made.

SUBSURFACE IRRIGATION

305. Description of Subsurface Irrigation Systems.—Subsurface irrigation has been used extensively in the United States since about 1870. As stated in Section 294, the process is limited to the disposal of water-carried wastes from isolated dwellings and the like. In its most primitive form, subsurface irrigation is practiced in the use of the so-called leaching cesspool illustrated

in Fig. 147. In any sizable installation, however, subsurface irrigation commonly consists of a series of pipes laid near the surface of the ground, with open joints so that the sewage may pass into the surrounding soil. A settling tank for retaining the settling solids, and a dosing tank are essential parts of such a plant. The dosing tank is provided so that the whole pipe system may be filled by the flush of the sewage, and thus prevent all the flow trickling out at a few joints as it would if the sewage came in a slow continuous flow. Intermittent discharge also permits the earth about the laterals to be drained and aerated between doses. Experience shows that the ground should not be shaded, if the best results are desired.

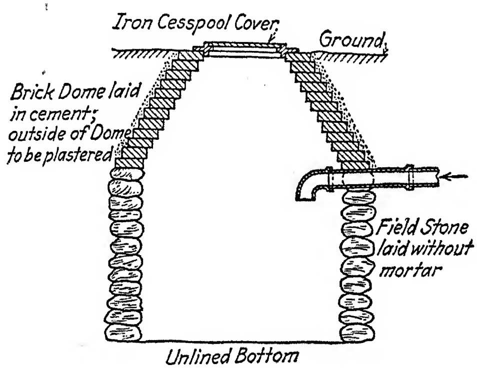


FIG. 147.—Leaching cesspool.

Subsurface irrigation systems are out of sight, generate no odors and operate satisfactorily in winter. On the other hand there is a tendency for the grass over the laterals to grow rank; the effluent may not be as good as from a sand filter, due to insufficient aeration; and if suspended matter and grease are allowed to pass into the pipe system it may have to be dug up and relaid after a few years, but this may not be serious in the case of small installations. A system of this kind with efficient settling basins should not require renovation for many years. Particular attention should be paid to preventing grease and other scum from leaving the tank. The equipment illustrated in Fig. 148 has been successful. The pollution of wells and other underground water supplies should be guarded against.

In heavy soils the irrigation tiles are, in some cases, laid 12 in. below the ground in trenches $3\frac{1}{2}$ to 4 ft. deep which are filled

with broken stone or cinders, underdrains being generally provided for collecting the effluent. In other cases large pits are dug and filled with sand, distributor pipes being laid near the top and underdrains being provided at the bottom. The surface of the pits is covered with loam and sod, and what is really a subsurface intermittent sand filter is thus formed.

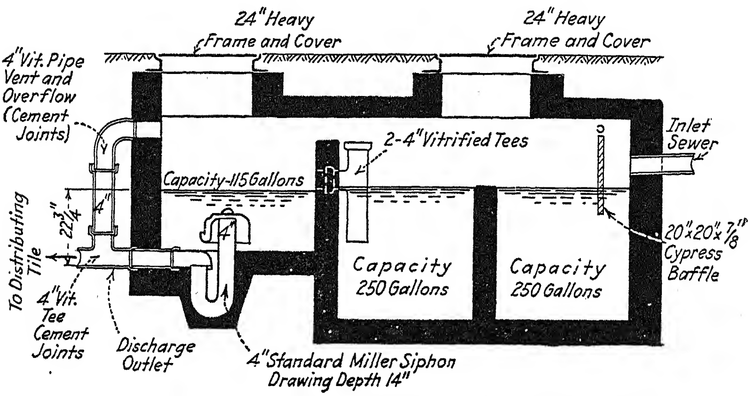


FIG. 148.—Settling tank and dosing tank for subsurface irrigation system.

306. Details of Subsurface Irrigation System.—Apart from their settling and dosing tanks, which are described in detail in Chapters XVI and XVII, subsurface irrigation systems consist of of open-jointed tile distributor pipes, sometimes fed by vitrified tile headers with tight joints. The irrigation pipes are usually 3 or 4 in. in diameter and are placed in rows 3 to 6 ft. apart. They are laid about 12 in. below the ground, but may vary in depth up to 24 in., to allow for unevenness of the surface. The tiles are each 1 or 2 ft. long and a space of about ¼ in. is left between them. The joints are surrounded by graded gravel, in the manner shown in Fig. 190. The length of distributing pipe necessary is generally estimated at 20 to 100 ft. per person connected, depending on the porosity of the soil. The slope of the irrigating pipes should be greater in porous soil than in more impervious material, so that sewage may flow to the lower end of the system and be discharged as uniformly as possible from all the openings. In general the slope should be from 2 to

5 in. per 100 ft. Two or more separate piping systems may be provided, to permit part of the irrigation area to rest after having been in service for some time, although this is not necessarily of advantage.

307. Theoretical Basis of Design.—In 1920 a committee of the American Public Health Association[1] made a report on rural sanitation, which includes a study of subsurface irrigation systems. The following outline giving a theoretical basis for the design of tile distribution systems follows the committee's report quite closely.

Area of Tile Field.—Assuming that the soil is highly compacted and that its grain size is that of very fine sand or silt with an effective size of 0.025 mm. and a uniformity coefficient of 2.5, Hazen's formula for the flow of water through sand (see Section 378), $v = cd^2\frac{h}{l}\left(\frac{T+10}{60}\right)$, indicates that the daily downward percolation of water corresponds to a depth of sewage of 4 in. over the surface. In this computation c is taken at the extremely low value of 200 and the temperature is assumed to be 40°F.

If a factor of safety of 5 be assumed, taking 0.8 in. of water daily, we arrive at a limit of $0.8 \times 144 \div 231 = 0.5$ gal. per square foot, which is the factor used. One-half gallon per square foot at 50 gal. per capita daily corresponds to 100 sq. ft. per capita. For a smaller per capita volume and correspondingly stronger sewage, increase the area per gallon by basing it upon 100 sq. ft. per capita. Thus is derived the first rule—*Area of Tile Field:* Not less than 100 sq. ft. per capita, nor less than 2 sq. ft. per gallon daily.

Length of Tile.—If a = area in sq. ft., p = population, l = length of tile in ft., and s = spacing of tiles in ft., then the area being fixed, length and spacing of tile are interdependent, for $a = pls$. It may be assumed that the liquid and colloidal organic matter permeates the entire area of the tile field for a greater or less depth, but solids which may be carried into the tile will move but a short distance away from the tile joints and be filtered out from the liquid which passes on by capillarity through the soil. It may be assumed that the suspended solids which are carried over into the tiles will not amount to more than 100 p.p.m. About one-third of these will be inorganic, and about two-thirds will be organic and will be partially taken up by vegetation or dissipated in the soil. At a specific gravity of 1.5 this becomes for 50 gal.

$$\frac{50 \times 100 \times 231}{1{,}000{,}000 \times 1.5} = 0.77 \text{ cu. in. per capita daily}$$

This material will be very fine and may, to some extent, be washed away from the joints into the earth by the flowing sewage, but neglecting this action and assuming that it fills the voids in the soil adjacent to the tile for a radius of 6 in., and that the voids amount to 20 per cent, there will be a cylinder 12 in. in diameter and 12 in. long surrounding each joint, from

[1] *Am. Jour. Pub. Health,* 1921; **11,** 551.

which must be taken the volume of the tile. For a 2-in. tile ½-in. thick the volume of voids in the hypothetical cylinder of earth will be

$$12 \times \pi(6^2 - 1\tfrac{1}{2}^2) \times 0.20 = 255 \text{ cu. in.}$$
$$255 \div 0.77 = 331 \text{ days per capita per joint.}$$

Applying a factor of safety of three gives 110.3 days per capita per joint. 6 years = 2,192 days. 2,192 ÷ 110.3 = 19.8. Hence use a length of tile of 20 ft. per capita and it may be expected that the plant will operate on the above basis for about 6 years.

If again it be assumed that the inorganic solids gradually collect in the tile and that the limit is reached when the tile is half filled, the following results: Volume of 2-in. tile 20 ft. long half full $= \pi \times 10 \times 12 = 377$ cu. in.

$$377 \div \frac{50 \times 231 \times 100}{3 \times 1.5 \times 1{,}000{,}000} = 1{,}469 \text{ days or } 4.02 \text{ years}$$

Spacing of Tile.—Having obtained the area and length of tile, the spacing between lines of tile is determined as follows:

$$s = a \div pl$$
$$l = 20.$$

whence

$$s = a \div 20p.$$

By the first rule above mentioned, a must be not less than 100 sq. ft. per capita $= 100p$, nor less than 2 sq. ft. per gallon daily $= 2pG$ where $G =$ gal. per capita daily. Using these values of a, the limiting minimum values are:

$$s = 100p \div 20p = 5$$
$$s = 2pG \div 20p = G \div 10.$$

Slope of Tile.—An efficient system operates equally from every joint, covering the entire area. A small drizzling flow will pass out at the first joint and, at best, discharge from but few: therefore provide a flushing siphon and make each discharge equal to the capacity of the distributing tile. This is important and often, when overlooked, the result is failure.

The time of filling the drains is short compared with the time of emptying, hence the discharge from the tile joints is under hydrostatic conditions.

In order to make the flow from all points equal, the head must be the same over the whole system, hence lay the tile level.[1]

Any collection of sediment in the tile would be apt to be carried on and piled up and the tile eventually plugged if operated under great velocity and high head.

For the longest system usually contemplated 20 lines of tile 100 ft. long will suffice and will readily fill from a common header. Five feet of 4-in. header will hold a volume equal to 20 ft. of 2-in. tile. Five-inch and 6-in. headers respectively will fill 31.25 ft. and 45 ft. of 2-in. tile.

Size of Tile and Number of Discharges Daily.—With the area, length and spacing determined, the size of tile is next to be found.

[1] It was previously stated that slopes of 2 to 5 in. per 100 ft. are commonly employed in practice. This is because the joints nearest the tank begin to discharge first, and will therefore discharge more than the more distant joints unless some slope is provided.

The area is based mainly on the amount of liquid handled, the length of tile upon the population contributing.

The amount of each discharge has been specified as equal to the capacity of all the tiles. Hence this capacity equals the total daily flow (Q in gal.) divided by the number of discharges per day (n), or algebraically as follows:

$$Q \div n = (\pi d^2 \times 12 \times 20p) \div (4 \times 231)$$

whence, if d = the diameter of the tile in inches,

$$d^2 = (4 \times 231Q) \div (\pi \times 12 \times 20np) = G \div 0.816n,$$

hence $d = 1.11\sqrt{G \div n}$ and $n = G \div 0.816d^2$

For $d = 2$ in., $n = G \div 3.264$

For $d = 3$ in., $n = G \div 7.344$

For $d = 4$ in., $n = G \div 13.056$

Plotting the values of n for various values of G in the last three equations there result three lines for $d = 2$ in., 3 in., and 4 in. (see Fig. 149).

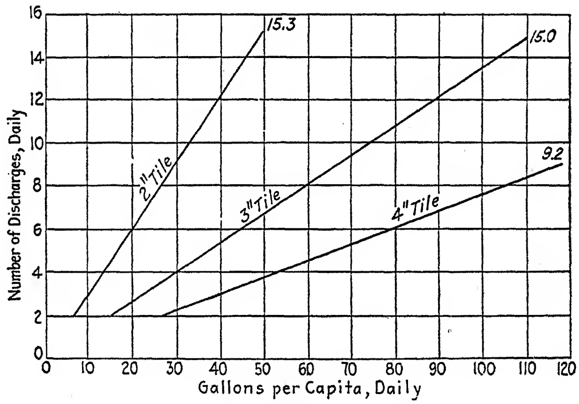


FIG. 149.—Capacity of subsurface irrigation tile.

Note that 9.2 discharges daily will not be exceeded in 4-in. tile until the per capita flow is over 120 gal. daily, that 3-in. tile with 15 discharges will carry 110 gal. per capita daily and that 2-in. tile does not discharge more than 15.3 times for 50 gal. per capita daily. With a 2-in. tile the daily limit of ½ gal. per square foot will fill 3.06 ft. of the tile, and at the prescribed spacing of 5 ft. there will be 5 × 3.06 = 15.3 discharges daily. Evidently the smaller diameter of tile is cheaper, as is also the smaller dosing chamber. If it be considered that about one-half the daily flow is concentrated in 8 busy hours, and if the discharges during this period be limited to one per hour, we shall have 16 in 24 hours. This may be taken as the limit and will confine most rural plants to 2-in. tile. It will be noted from the diagram that 3-in. tile will not exceed 11 doses per day at 80 gal. per capita per day.

Thus the second rule is derived:

Tile Distributors—

(*a*) Length, 20 ft. per capita.

(*b*) Spacing, gal. per capita ÷ 10, but not less than 5 ft.

(*c*) Diameter, 2 in. up to 50 gal. per capita and 16 discharges daily; 3 in. above 50 gal. per capita daily.

From which follows the third rule:

Number of discharges per day = daily flow ÷ contents of tile, but not over 16.

It will be noted that this system depends upon the porosity of the soil and its drainability. If it is very open a smaller area might be used and the length of tile per capita reduced proportionally. The diameter and capacity, however, are dependent upon the number of discharges per day. A larger tile will hold a larger deposit of solids before it is choked, but efficient sedimentation should be provided in advance, and it must be remembered that it may be necessary to relay the tile at intervals of 4 to 6 years, although there are cases where a system has operated successfully for 12 years.

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Problems

1. Given an American city of 10,000 people with sewage flow of 115 gal. per capita daily. (*a*) Assuming that disposal is to be by ridge-and-furrow irrigation with little cropping, and that the available land has soil of texture similar to that at the Berlin sewage farms, what area will be required based on the conclusions of the Royal Commission? (*b*) If 15 per cent of the area is taken up by roads and only 50 per cent of the remainder is under irrigation at any one time, what is the net rate of filtration?

Ans. (*a*) 38 acres.
(*b*) 71,000 gal. per acre daily.

2. Given a country home occupied by 15 persons and a sewage flow of 50 gal. per capita daily, design a subsurface irrigation system to dispose of the

water-carried wastes. Prepare a sketch showing the arrangement of the irrigation tiles and feeders.

3. Consider that the city-owned irrigation farms of Paris took their prorata share of sewage disposed of by irrigation in 1923 on areas as in 1924; that interest charges upon capital investment were 4 per cent, and that rentals were \$24 per acre, with total cost and operating charges as given for 1900. What was the net cost per million gallons of disposing of sewage by irrigation? *Ans.* \$77 per m.g.

CHAPTER XIV

PRINCIPLES OF SEWAGE TREATMENT

308. Treatment Processes.—As shown in Sections 266 and 267 of Chapter XI, sewage treatment may involve (1) the separation of the solid sewage matters from the liquid sewage; (2) treatment of the solid matters; and (3) treatment of the liquid sewage. Since the methods by which sewage treatment is accomplished are many, while the fundamental principles of operation are relatively few, it will be best, before proceeding to a discussion of sewage treatment processes, to study the theoretical principles that underlie these processes. Confining ourselves to the most important principles we shall have to consider (1) in connection with the separation of solid matters and liquid sewage: the principles of sedimentation and chemical precipitation; (2) relative to the treatment of the solid matter: the principles of sludge digestion; and (3) as regards treatment of the liquid sewage: the principles of filtration, oxidation and disinfection.

Treatment processes such as screening and flotation, the theory of which is either obvious (screening) or of limited significance in the field of sewage treatment (flotation), will not be included.

SEDIMENTATION

309. Sedimentation Defined.—Sedimentation is the process by which suspended matter in sewage subsides and is deposited by gravity. While screening will remove the larger solids floating or suspended in sewage, it is often desirable to rid the sewage of a greater proportion of the suspended mineral and organic solids before discharging it into natural water courses or before subjecting it to filtration or other treatment processes. If these solids are not removed they may form offensive sludge deposits and increase the oxygen demand of the waters receiving them. In treatment works they tend to clog fine-grained filters and to impose an unnecessary burden on coarse-grained filters. In the disinfection of sewage they are not penetrated by the disinfecting

chemicals, and the efficiency of the process is thus reduced. The removal of suspended matter is, therefore, an important function of sewage treatment works.

In general, distinction is made between the removal of heavy mineral solids called *grit*, which is commonly accomplished in *grit chambers* or *detritus tanks*, and the settling as *sludge* of lighter, more largely organic, sewage solids in *sedimentation tanks*.

310. Subsidence of Suspended Particles.—Sewage matters in suspension are found in this state either because their specific gravity is less than that of water or because the velocity of flow is such that they are carried along by the sewage. Since the transporting capacity varies as the sixth power of the velocity, it follows that a reduction in velocity will cause the sewage to deposit part of its burden of solid matter under the influence of gravity. Apart from substances that are lighter than water and will rise to the surface (oil, grease and floating matters), some of the suspended solids (sandy matters or grit) are deposited promptly when the velocity is reduced below that normally obtaining in sewers; others (settling solids) require lower velocities and longer periods of time for settling; still others (colloidal and non-settling solids) will not subside within a reasonable time even when the sewage is perfectly quiescent.

The processes governing sedimentation are extremely complex. In order to approach them it is necessary first to restrict discussion to some of the elementary considerations of the problem. Among these the subsiding characteristics of particles in still water must be given first attention.

Under the influence of gravity all particles heavier than water tend to settle. This tendency, however, is opposed by (*a*) the frictional resistance of the liquid and (*b*) its resistance to deformation or viscosity. While both are effective at the same time, experiments have shown that for larger particles friction controls, whereas for small ones viscosity controls.

The factors governing the rate of subsidence of particles in still water are: (1) the specific gravity of the particles relative to the liquid; (2) their size and shape, which together govern the ratio of volume to surface area; (3) the temperature of the liquid, which controls its viscosity as well as its density.

Assuming the particles to be spherical in shape, their weights and volumes vary as the cubes of their diameters while their surfaces vary as the squares. Hence the relative surface pre-

sented by small particles is much greater than for large ones, and small particles settle much more slowly than do large ones.

311. Stokes' Law.—Stokes has formulated the relation that exists between the rate of subsidence of very small particles and the various factors influencing their sedimentation. Expressing the values in the C.G.S. system, Stokes' law is formulated as follows:

$$v = \frac{2}{9}g\frac{s - s'}{e}r^2 \text{ where}$$

v = velocity of subsidence
g = acceleration due to gravity
s = density of the particle
s' = density of the liquid
e = viscosity of the liquid
r = radius of the particle

Since viscosity is governed by temperature the viscosity factor may be replaced by a temperature factor. It will be found that the ratio of the viscosity at any temperature to that at 50°F. ($e = 0.01303$) is approximately equal to $\frac{60}{T + 10}$ where T is the temperature in °F. Making this substitution, expressing v in mm. per sec., and replacing r by d, the diameter in mm., it is found that

$$v = 418(s - s')d^2\frac{T + 10}{60}$$

For quartz sand of specific gravity 2.65 the formula becomes:

$$v = 690d^2\frac{T + 10}{60}$$

For sewage solids of specific gravity 1.2:

$$v = 84d^2\frac{T + 10}{60}$$

Stokes' law holds for particles smaller than 85μ (0.085 mm.) in diameter. Beyond this limit viscosity, while still active, no longer controls, and friction comes more and more into play, the velocity varying as the square root of the diameter when friction controls. There is a transition space in between. This has been investigated by Hazen for quartz sands (sp. gr. 2.65) with diameters of 0.1 to 1 mm. Hazen's values may be formulated approximately as follows:

$$v = 100d\frac{T + 10}{60}$$

Assuming that the specific gravity of the material enters into the equation as in Stokes' law, the relation for particles of specific gravity 1.2 is $v = 12d\frac{T + 10}{60}$.

The rate of subsidence of particles expressed in mm. per sec. is known as their *hydraulic subsiding value.* The hydraulic subsiding values of particles with specific gravity 2.65 (quartz sand) and 1.20 (sewage solids) are plotted in Fig. 150. They hold for a temperature of 50°F. For other temperatures they must be multiplied by the temperature factor $\frac{T + 10}{60}$.

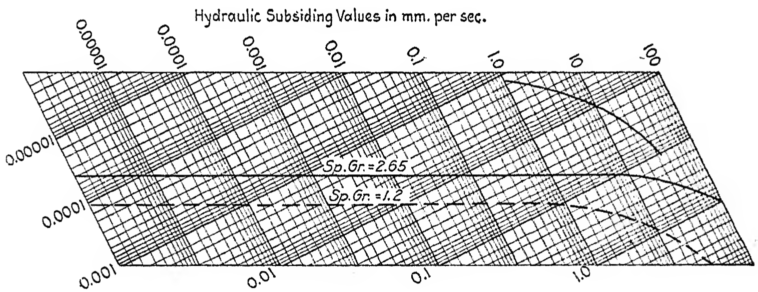

Fig. 150.—Hydraulic subsiding values of particles with specific gravity 2.65 and 1.20 in still water at 50°F. (10°C.).

The very marked effect of temperature upon subsidence should be noted carefully. Due to it much greater sedimentation efficiencies can be obtained in summer than in winter. At a temperature of 74°F., for example, the rate of settling is twice as great as at the freezing point.

312. Hazen's Theory of Sedimentation.—Hazen[1] has made a careful mathematical study of sedimentation, and the student should familiarize himself with the original paper from which the following paragraphs are abstracted for the purpose of showing the various factors entering into the problem.

In order to simplify the approach, Hazen made the following assumptions:

1. Whenever a particle of suspended matter strikes the bottom it remains where it strikes and is never carried forward on the bottom or picked up again.

[1] *Trans.*, A.S.C.E., 1904; **53**, 45.

2. All particles have the same hydraulic subsiding value, v.

Let t = time for a particle to fall from the surface to the bottom of the basin, the water meanwhile being absolutely still.

a = time of sedimentation in case the action is intermittent; and, in case of continuous operation, let a be the quotient obtained by dividing the capacity c of the basin by e, the quantity of water entering or leaving it during each unit of time.

n = number of basins, in case several basins are used in series.

x = the proportion of sediment remaining at the end of the process, the amount at the beginning being taken as unity (1).

Proposition 1. *Intermittent Basin.*—Assuming that the basin is full of water and absolutely quiet:

$$x = 1 - 1 \times \frac{a}{t}$$

The values obtained from this expression are plotted as "Theoretical maximum" in Fig. 151. They represent the theoretical maximum sedimentation that can be obtained in an intermittent basin; they cannot be reached in practice. Adverse factors are: (1) the kinetic energy of the entering water which is still capable of producing vortex motion after long periods of apparent quiescence; (2) the action of wind; (3) changes in temperature which, though slight, set up convection currents. All of these induce mixing of the liquid.

Proposition 2. *Intermittent Basin.*—Assuming that the basin is full as in proposition 1, but that the water is kept mixed during the process of sedimentation to such an extent that the density of the suspended matter is the same in all parts of the basin, the amount of suspended matter remaining after an interval of time da is:

$$x_1 = 1 - 1 \times \frac{da}{t} = \left(1 - \frac{da}{t}\right)$$

After another interval da:

$$x_2 = x_1 - x_1 \frac{da}{t} = \left(1 - \frac{da}{t}\right)^2$$

Since there are $\frac{a}{da}$ intervals:

$$x = \left(1 - \frac{da}{t}\right)^{\frac{a}{da}}$$

Developing this expression binomially and neglecting all values containing the first derivative da, $x = 0.367878$ for $\frac{a}{t} = 1$; other determinations can be made similarly. The values obtained are plotted as "Intermittent Basin (Theoretical)" in Fig. 151. They represent the theoretical minimum sedimentation for an intermittent basin. For n intervals of time, $x = \left(1 - \frac{a}{nt}\right)^n$.

Proposition 3. *Single Continuous Basin.*—Assuming that the water enters and leaves the basin continuously and that the density of the suspended matter is kept the same in all parts of the basin by mixing, the amount of sediment deposited in the time a is $(1 - x)$; in the time da it is $\frac{da}{a}(1 - x)$.

Since the time required to deposit all the particles is t, the proportion deposited in the time da is $\frac{da}{t}$ and the amount, $x\frac{da}{t}$. Hence

$$x\frac{da}{t} = \frac{da}{a}(1 - x)$$

$$x = \frac{1}{1 + \frac{a}{t}}$$

The values obtained from this expression are plotted as "One Basin Continuous" in Fig. 151.

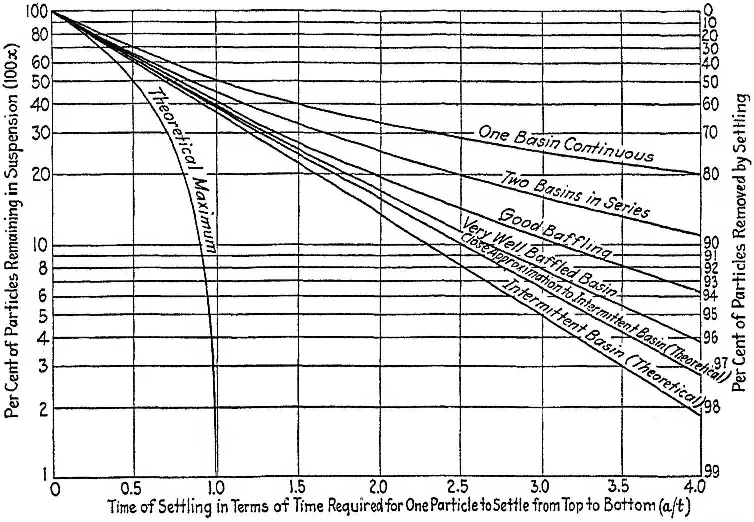

FIG. 151.—Sedimentation obtained in differently arranged settling units. (*After Hazen.*)

Proposition 4. *Two Basins in Series.*—Assuming that two basins are so arranged that the water from one enters the second, all other conditions remaining the same as in proposition 3, the time of sedimentation in each basin is $\frac{a}{2}$. Hence from proposition 3, the amount of suspended matter remaining after passage through the first basin is $x_1 = \frac{1}{1 + \frac{a}{2t}}$ and the amount remaining after the second is $x = \frac{x_1}{1 + \frac{a}{2t}} = \frac{1}{\left(1 + \frac{a}{2t}\right)^2}$. The values obtained are plotted as "Two Basins in Series" in Fig. 151.

Proposition 5. *More than Two Basins in Series.*—Extending the reasoning of proposition 4 to n basins in series, $x = \frac{1}{\left(1 + \frac{a}{nt}\right)^n}$. The values for

$n = 4$, 8 and 16 are plotted as "Good Baffling," "Very Well Baffled Basin," and "Close Approximation to Intermittent Basin," respectively, n Fig. 151. For $n = \infty, x = \dfrac{1}{\left(1 + \dfrac{a}{nt}\right)^n} = \left(1 - \dfrac{a}{nt}\right)^n$, which is the same expression as that of proposition 2, shown as "Intermittent Basin (Theoretical)" in Fig. 151. In other words, theoretically, a sedimentation basin operated on the continuous system with absolutely complete baffling would give the same results as a basin on the intermittent system kept absolutely mixed from top to bottom.

Proposition 6. *Very Shallow Basin.*—If a very shallow basin be assumed, with area b, depth h and capacity c, and e is the quantity of water treated in a unit of time, while v is the hydraulic subsiding value of the particles,

$$a = \frac{c}{e} = \frac{bh}{e}$$

$$t = \frac{h}{v}$$

$$\frac{a}{t} = \frac{bv}{e}$$

It will be noted that a/t, which is the only factor entering into the expression measuring sedimentation efficiencies of the different types of basins, is independent of the depth of the basin, but varies directly with the area of the basin and the hydraulic subsiding value of the particles and inversely with the quantity of water treated in a unit of time. This fact that, within limits to be discussed later, sedimentation is independent of depth, has of late years been put to practical use by Imhoff in the construction of what he has called *shallow underdrained settling basins* which act at the same time as sludge drying beds (see Chapter XVI).

Proposition 7. *Basin with Horizontal False Bottoms.*—Assuming that a basin is divided by horizontal plates into two or more compartments one above the other, each horizontal subdivision will provide a surface to receive sediment. Since sedimentation is independent of depth and merely dependent upon surface area, the sedimentation efficiency will be increased in proportion to the increase in area. The most serious practical difficulty to be met in carrying out this idea is the increased cleaning necessitated. The *slate beds* of Dibdin operated on this principle; expansion of this principle explains in part the sedimentation efficiency of *contact beds* and *intermittent sand filters* (see Section 377).

Proposition 8. *Suspended Particles Having Unequal Hydraulic Subsiding Values.*—Since a is the same no matter what the size of the particle, while t varies for particles of different size and specific gravity, the per cent of particles of each size and weight removed in a given basin will vary, a greater proportion of small and light particles remaining in the effluent than of large and heavy ones. This permits of *differential sedimentation* as applied in grit chambers, in which it is the object to remove the larger mineral solids while permitting the lighter organic ones to escape (see Section 314).

313. Limitations of Hazen's Theory.—In applying Hazen's theory of sedimentation to settling basins for sewage, a number of limitations of the theory must be kept in mind, most of which are pointed out in the original paper.

Bottom Velocities.—It was assumed that any particle striking the bottom remains there. This is not necessarily so. Both horizontal and vertical currents operate against it. The vertical currents are in the nature of vortex motion, wind induced currents and temperature convection currents. The horizontal currents are chiefly due to the horizontal movement of the water through continuous basins. It would seem that, apart from adhesion, which may be a factor of considerable importance in the case of sewage solids, a bottom velocity equal to the hydraulic subsiding value would prevent deposition or lift the particle or move it along the bottom. Practically nothing is known about the magnitude of bottom velocities in relation to mean horizontal velocities. The data at hand seem to show that mean velocities of 20 to 40 times v are required to move sediment along the bottom. Since it appears from Fig. 151 that a/t must reach a value of about 1.5 to secure satisfactory sedimentation, say 60 to 80 per cent removal, it follows that the use of ratios of basin length to depth of less than 30:1 to 60:1 is indicated.[1] To this extent then sedimentation is not independent of depth as shown in proposition 6.

Granular and Flocculent Solids.—Depth enters into the problem of sedimentation in another way also. Much of the suspended matter in sewage is flocculent in character, *i.e.* the particles tend to gather together into aggregates. Sweeping down through the sewage they enmesh other particles, become larger in size and settle more rapidly. This is not so much the case with the granular solids found in turbid river waters. The process of flocculation explains the efficiency of some of the deep basins used in sewage treatment and of artificial aggregation by the addition of electrolytes or chemical coagulants in sedimentation combined with *chemical precipitation.* Salt water, too, induces flocculation. The rapidity of sedimentation observed when sewage is discharged into tidal water must be ascribed to this fact; for it cannot be supposed that an individual particle would

[1] If l = length of basin and V = mean velocity,

$$V = \frac{l}{a};\ v = \frac{h}{t};\ \frac{l}{h} = \frac{aV}{tv}$$

settle more rapidly in salt water than in fresh water of lower density and viscosity.

Detention Period.—The time of sedimentation a is commonly called the detention period. In practice it is generally expressed in hours. If, for example, the rate of sewage flow is 2.4 m.g.d. and the tank capacity is 100,000 gal., the detention period is $\frac{0.1 \times 24}{2.4} = 1$ hour. The *flowing-through period,* on the other hand, is the time actually required for a unit volume of sewage to pass from inlet to outlet. Theoretically the detention period and the flowing-through period are one and the same measure. Actually they will vary. Displacement of the sewage in the

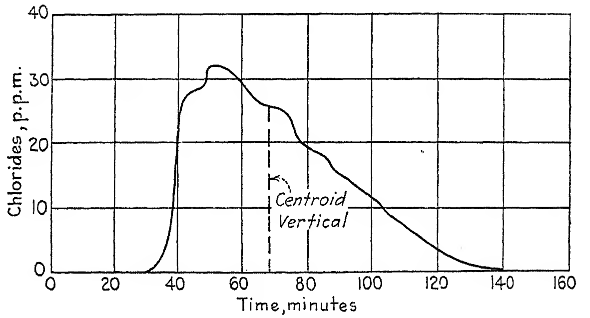

FIG. 152.—Flowing-through period of sedimentation tank at Phillipsburg, N. J

tank by the incoming sewage is seldom uniform, depending upon the distribution of the incoming sewage, the velocities in the tank, the method of baffling and the way in which the effluent is withdrawn. As a result part of the sewage reaches the outlet more rapidly than does the remainder and part stays in the tank for a longer time. The percentage ratio of the flowing-through period to the detention period is, therefore, a measure of the efficiency of distribution of the sewage. The best results are obtained where each unit of sewage flows through the basin with the least mixing with preceding and succeeding units.

The flowing-through period is readily determined by adding a solution of commercial salt to the inflowing sewage and titrating the effluent to determine the amount of chlorides it contains. After the salt first reaches the effluent there will be a gradual increase in chlorides until a maximum is reached, after which there will be a gradual decrease. A typical curve obtained by

Capen[1] for the tanks at Phillipsburg, N. J., is shown in Fig. 152. The flowing-through period is fixed by the location of the vertical gravity axis of the area under the curve. In the case of Phillipsburg the flowing-through period was found to be 1.17 hours against a detention period of 2.42 hours. The flowing-through period was therefore 48 per cent of the detention period. It should be noted that the peak concentration of chlorides occurred before the flowing-through period was completed, and since this peak commonly coincides with the maximum color produced by dyes added to the sewage it is evident that the use of dyes to determine the flowing-through period is not reliable. Electrical resistance measurements can be substituted for chloride titration.

Summary.—With present knowledge of the sedimentation characteristics of sewage solids, Hazen's theory can find only limited application. It holds in particular for the settling of heavy gritty substances, but is useful in other problems of sedimentation by showing that (1) within limits the area of a settling basin is more important than its depth; (2) values of a/t of 1.5 or more should be obtained to secure good basin efficiency; (3) the ratio of length to depth should be limited; (4) good baffling is of prime importance and will increase the efficiency by about 25 per cent; (5) temperature is of great significance; (6) wind action should be reduced to a minimum.

314. Application of Hazen's Theory.—To illustrate the application of Hazen's theory of sedimentation let it be required to investigate theoretically the performance of the grit chambers at the Irondequoit Plant of the Rochester, N. Y., sewerage system. A plan and a longitudinal section of one of the six chambers are shown in Fig. 153.

Assuming a velocity of 1 ft. per second, a freeboard of 0.5 ft. and a deposit of grit of 3 ft., the characteristics of each chamber are as follows:

Area b = 900 sq. ft.

Effective depth h = 5.5 ft.

Length l = 90 ft.

Ratio of length to depth $\frac{l}{h} = \frac{90}{5.5} = 16.4$.

Mean velocity of flow V = 1 ft. per second.

[1] *Eng. News-Record*, 1927; **99**, 836.

Detention period $a = 90/1 = 90$ sec.
Rate of sewage flow $e = 1 \times 5.5 \times 10 = 55$ c.f.s.

$$\frac{a}{t} = \frac{bv}{e} = \frac{900 \times v}{55 \times 304.8^1} = 0.0537v$$

Since this is a continuous basin without baffling, the highest line in Fig. 151 applies. Taking the values of a/t corresponding to certain percentages of removal of suspended matter from the curve, or computing them from the formula $x = \dfrac{1}{1 + \frac{a}{t}}$, the findings can be arranged in tabular form as below.

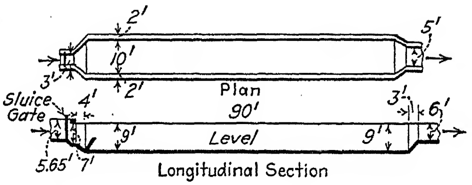

Fig. 153.—Grit chamber at Irondequoit Plant, Rochester Sewage Works. (*After Skinner.*)

Computations for $v = \dfrac{1}{0.0537}\dfrac{a}{t}$ are included (Column 3), and the diameters of particles removed, corresponding to the various hydraulic subsiding values (Columns 4 and 5) as taken from Fig. 150, are added.

Suspended matter removed, per cent	$\frac{a}{t}$	Hydraulic subsiding value v, mm. per second	Diameter of particle, mm.		Suspended matter remaining, per cent
			Mineral, sp. gr. 2.65	Organic, sp. gr. 1.2	
10	0.11	2.0	0.05	0.2	90
20	0.25	4.7	0.08	0.4	80
30	0.43	8.0	0.12	0.7	70
40	0.67	12.5	0.14	1.2	60
50	1.00	18.6	0.18	2	50
60	1.50	28.0	0.27	4	40
70	2.33	43.4	0.43	>10	30
75	3.00	55.9	0.56	....	25
80	4.00	74.5	0.75	....	20
85	5.67	106	1.1	....	15
90	9.00	168	2.2	....	10
95	19.00	354	>10	....	5
99	99.00	1,850	....	....	1

Assuming that the theoretical results measure conditions with a fair degree of accuracy, the differential action of a grit chamber of this type is

[1] To convert to millimeters.

quite apparent. While more than 50 per cent of the mineral matter 0.2 mm. in diameter is removed, only 10 per cent of the organic matter of this size is deposited. While 90 per cent of the mineral matter 2 mm. in diameter settles, only 50 per cent of the organic matter of this size is removed.

Instead of working problems in this way it is possible to develop approximate formulas for use. Proceeding from the equation $a/t = bv/e$, and expressing v in terms of d as in Section 311, the following formulas are obtained:

1. For particles smaller than about 0.1 mm.

(a) specific gravity 2.65; $d = 0.665\left(\frac{a}{t} \times \frac{e}{b} \times \frac{60}{T + 10}\right)^{1/2}$

(b) specific gravity 1.2; $d = 1.90\left(\frac{a}{t} \times \frac{e}{b} \times \frac{60}{T + 10}\right)^{1/2}$

2. For particles between 0.1 and 1 mm.

(a) specific gravity 2.65; $d = 3.05 \times \frac{a}{t} \times \frac{e}{b} \times \frac{60}{T + 10}$

(b) specific gravity 1.2; $d = 25.2 \times \frac{a}{t} \times \frac{e}{b} \times \frac{60}{T + 10}$

In these formulas e/b represents the c.f.s. of sewage treated per square foot of surface, and the values of a/t for each type of sedimentation unit must be taken from the curves in Fig. 151 or from the corresponding equations of Section 312.

CHEMICAL PRECIPITATION

315. Chemical Precipitation Defined.—Chemical precipitation is a method of increasing the sedimentation of suspended matter and inducing that of colloidal matter by the addition to the sewage of chemicals that in one way or another form *floc* in the liquid, the floc drawing to itself the substances that it is desired to remove or being produced by them. In water purification this process is called *coagulation.*

Many different substances have been used as precipitants. The most common ones are calcium oxide or lime, aluminum sulphate or alum, lime and ferrous sulphate or copperas, sulphuric acid and sulphur dioxide. The degree of clarification obtained depends upon the quantity of chemicals used and the care with which the process is controlled. It is possible by chemical precipitation to obtain a clear effluent, substantially free from matter in suspension or in the colloidal state. From 80 to 90 per cent of the total suspended matter can be removed,

from 50 to 55 per cent of the total organic matter, and from 80 to 90 per cent of the bacteria. These figures compare very favorably with plain sedimentation in which from 50 to 70 per cent of the total suspended matter and from 30 to 35 per cent of the organic matter settle out.

The handling and disposal of the sludge resulting from chemical precipitation is one of the greatest difficulties of this method of treatment. Sludge is produced in great volume, often reaching 0.5 per cent of the volume of sewage treated. Furthermore, although the effluent may have a moderately satisfactory appearance, it is ordinarily putrescible and not comparable with the effluents produced by oxidation processes. These drawbacks, together with the expense of the chemicals, have curtailed the use of chemical precipitation and have led to its abandonment in a number of municipal plants formerly employing the process. The greatest field for usefulness seems to be in the treatment of industrial wastes. The use of sulphuric acid for the precipitation of grease, as practiced at Bradford, England, for the municipal sewage which is rich in wool-scouring wastes, provides one of the few cases in which a substantial return is had from the sludge resulting from chemical precipitation.

316. Reactions Involved in Chemical Precipitation.—The coagulants added to sewage in chemical precipitation react either with substances normally present in the sewage or with substances added for this purpose. The quantity of chemical employed is commonly expressed in grains per gallon or in pounds per million gallons. One grain per gallon equals 17.1 p.p.m., and since 1 p.p.m. equals 8.33 lb. per million gallons, one grain per gallon is equivalent to 142.5 lb. per million gallons.

The reactions involved in chemical precipitation will be taken up in the following order: (1) alum; (2) copperas and lime; (3) lime; (4) sulphuric acid and sulphur dioxide.

1. *Alum.*—When alum is added to sewage containing in solution calcium and magnesium bicarbonate alkalinity, the reaction that occurs may be illustrated as follows:

$$\underset{\substack{\textit{Aluminum sulphate.}\\ \text{Soluble.}}}{\overset{666.7}{Al_2(SO_4)_3 18H_2O}} + \underset{\substack{\textit{Calcium bicarbonate.}\\ \text{Soluble.}}}{\overset{3 \times 162\,=\,3 \times 100\ CaCO_3}{3Ca(HCO_3)_2}} \rightleftarrows \underset{\substack{\textit{Calcium sulphate.}\\ \text{Soluble.}}}{\overset{3 \times 136}{3CaSO_4}} +$$

$$\underset{\substack{\textit{Aluminum hydroxide.}\\ \text{Insoluble.}}}{\overset{2 \times 78}{2Al(OH)_3}} + \underset{\substack{\textit{Carbon dioxide.}\\ \text{Soluble.}}}{\overset{6 \times 44}{6CO_2}}$$

The insoluble aluminum hydroxide is formed as a bulky, gelatinous floc which settles slowly through the sewage, sweeping out suspended matter and producing other changes which will be described in Section 317. The reaction is exactly analogous when magnesium bicarbonate is substituted for the calcium salt. The numbers above the chemical formulas are the combining weights of the different substances and denote therefore what quantity of each is involved. Since alkalinity is reported in terms of calcium carbonate ($CaCO_3$) the molecular weight of which is 100, the quantity of alkalinity required to react with 1 grain per gallon of alum is $17.1 \times \frac{3 \times 100}{666.7} = 7.7$ p.p.m. If less than this amount is available for decomposing the alum, artificial alkalinity must be added. This is seldom required in sewage treatment. Lime is commonly used for this purpose where necessary.

2. *Copperas and Lime.*—In ordinary sewages, copperas cannot be employed alone as a precipitant and lime must be added at the same time, as will appear from the reactions involved. The process is commonly spoken of as the *iron and lime process.*

$$\underset{\substack{\textit{Ferrous sulphate.}\\ \text{Soluble.}}}{\overset{278}{FeSO_4 7H_2O}} + \underset{\substack{\textit{Calcium bicarbonate.}\\ \text{Soluble.}}}{\overset{100CaCO_3}{Ca(HCO_3)_2}} \rightleftarrows \underset{\substack{\textit{Ferrous bicarbonate.}\\ \text{Slightly soluble.}}}{\overset{178}{Fe(HCO_3)_2}} + \underset{\substack{\textit{Calcium sulphate.}\\ \text{Soluble.}}}{\overset{136}{CaSO_4}} + \overset{7 \times 18}{7H_2O}$$

If lime, CaO, is now added in the form of *milk of lime,* $Ca(OH)_2$, [$CaO + H_2O \rightleftarrows Ca(OH)_2$] the reaction that takes place is:

$$\underset{\substack{\textit{Ferrous bicarbonate.}\\ \text{Slightly soluble.}}}{\overset{178}{Fe(HCO_3)_2}} + \underset{\substack{\textit{Calcium hydroxide.}\\ \text{Slightly soluble.}}}{\overset{2 \times 56\ CaO}{2Ca(OH)_2}} \rightleftarrows \underset{\substack{\textit{Ferrous hydroxide.}\\ \text{Very slightly soluble.}}}{\overset{89.9}{Fe(OH)_2}} + \underset{\substack{\textit{Calcium carbonate.}\\ \text{Very slightly soluble.}}}{\overset{2 \times 100}{2CaCO_3}} + \overset{2 \times 18}{2H_2O}$$

The ferrous hydroxide is next oxidized to ferric hydroxide, the final form desired, by the oxygen dissolved in the sewage:

$$\underset{\substack{\textit{Ferrous hydroxide.}\\ \text{Very slightly soluble.}}}{\overset{4 \times 89.9}{4Fe(OH)_2}} + \underset{\substack{\textit{Oxygen.}\\ \text{Soluble.}}}{\overset{32}{O_2}} + \overset{2 \times 18}{2H_2O} \rightleftarrows \underset{\substack{\textit{Ferric hydroxide.}\\ \text{Insoluble.}}}{\overset{4 \times 106.9}{4Fe(OH)_3}}$$

The insoluble ferric hydroxide is formed as a bulky, gelatinous floc similar to the alum floc. One grain per gallon of copperas requires $17.1 \times \frac{100}{278} = 6.2$ p.p.m. of alkalinity; $17.1 \times \frac{2 \times 56}{278} =$ 6.9 p.p.m., or 0.40 grain per gallon, of lime; and $17.1 \times \frac{32}{4 \times 278} =$

0.49 p.p.m. of oxygen. Oxidation is favored by a high pH value which is established to some extent by the lime.

In sewage treatment, lime is commonly added in excess of the amount required to complete the "iron and lime" reaction, the excess lime being relied upon as an additional clarifying agent in accordance with the principles set forth later in this section. Experience has shown that the best results are obtained when sufficient lime is added to produce a pink color when phenolphthalein is used as an indicator. Since the formation of ferric hydroxide is dependent upon the presence of dissolved oxygen, iron and lime cannot well be used with septic sewages or industrial wastes devoid of oxygen.

Ferric sulphate might take the place of ferrous sulphate and its use would often avoid the addition of lime and the requirement of dissolved oxygen. It has not been employed in the past because it was not readily obtainable as a commercial product; recently it has been placed on the market.

3. *Lime.*—When lime alone is added as a precipitant or is used in excess of the amount required for the precipitation of the iron in the iron and lime process, the principles of clarification are explained by the following reactions:

$$\underset{\substack{\textit{Calcium hydroxide.}\\ \text{Slightly soluble.}}}{\overset{56CaO}{Ca(OH)_2}} + \underset{\substack{\textit{Carbonic acid.}\\ \text{Soluble.}}}{\overset{44CO_2}{H_2CO_3}} \rightleftarrows \underset{\substack{\textit{Calcium carbonate.}\\ \text{Very slightly soluble.}^1}}{\overset{100}{CaCO_3}} + \overset{2 \times 18}{2H_2O}$$

$$\underset{\substack{\textit{Calcium hydroxide.}\\ \text{Slightly soluble.}}}{\overset{56CaO}{Ca(OH)_2}} + \underset{\substack{\textit{Calcium bicarbonate.}\\ \text{Soluble.}}}{\overset{100CaCO_3}{Ca(HCO_3)_2}} \rightleftarrows \underset{\substack{\textit{Calcium carbonate.}\\ \text{Very slightly soluble.}}}{\overset{2 \times 100}{2CaCO_3}} + \overset{2 \times 18}{2H_2O}$$

A sufficient quantity of lime, therefore, must be added to combine with all the free carbonic acid and with the carbonic acid of the bicarbonates (half-bound carbonic acid) to produce calcium carbonate which acts as the coagulant. Much more lime is generally required when it is used alone than when sulphate of iron also is employed. Where industrial wastes introduce mineral acids or acid salts into the sewage, these must be neutralized before precipitation can take place. In some cases the sewage contains iron wastes which may then take the place of the copperas introduced artifically in the iron and lime process. The student should figure from the above reactions how much

[1] The solubility of $CaCo_3$ is about 11 p.p. m.

free and half-bound carbon dioxide are neutralized by one grain per gallon of lime.

If too much lime is used in the treatment of sewage, some of the suspended organic matter will be dissolved by the caustic calcium hydroxide and the effluent may be worse than the untreated sewage. If insufficient quantities are added the effluent will not be well clarified.

4. *Sulphuric Acid and Sulphur Dioxide.*—Neither sulphuric acid nor sulphur dioxide is a floc-forming substance. When sulphuric acid is added to sewage it neutralizes the alkalinity as follows:

$$\begin{array}{ccccccccc} 98 & & 100\mathrm{CaCO_3} & & 136 & & 2\times 44 & & 2\times 18 \\ \mathrm{H_2SO_4} & + & \mathrm{Ca(HCO_3)_2} & \rightleftarrows & \mathrm{CaSO_4} & + & \mathrm{2CO_2} & + & \mathrm{2H_2O} \\ \textit{Sulphuric acid.} & & \textit{Calcium bicarbonate.} & & \textit{Calcium sulphate.} & & \textit{Carbon dioxide.} & & \\ \text{Soluble.} & & \text{Soluble.} & & \text{Soluble.} & & \text{Soluble.} & & \end{array}$$

Any excess remains in solution as sulphuric acid. Only soluble substances are formed and the mechanism of precipitation is, therefore, not due to the production of a chemical precipitate as in the previous instances. Precipitation must be laid to other causes which will be decribed in Section 317.

When sulphur dioxide is used it hydrolizes to form sulphurous acid and reacts with the alkalinity of the sewage to form bisulphites. Both of these compounds are oxidized in the presence of dissolved oxygen. The reactions may be stated as follows:

$$\begin{array}{ccccccccc} 2\times 82 & & 100\mathrm{CaCO_3} & & 202 & & 2\times 44 & & 2\times 18 \\ \mathrm{2H_2SO_3} & + & \mathrm{Ca(HCO_3)_2} & \rightleftarrows & \mathrm{Ca(HSO_3)_2} & + & \mathrm{2CO_2} & + & \mathrm{2H_2O} \\ \textit{Sulphurous acid.} & & \textit{Calcium bicarbonate.} & & \textit{Calcium bisulphite.} & & \textit{Carbon dioxide.} & & \\ \text{Soluble.} & & \text{Soluble.} & & \text{Soluble.} & & \text{Soluble.} & & \end{array}$$

$$\begin{array}{ccccc} 64 & & 18 & & 82 \\ \mathrm{SO_2} & + & \mathrm{H_2O} & \rightleftarrows & \mathrm{H_2SO_3} \\ \textit{Sulphur dioxide.} & & & & \textit{Sulphurous acid.} \\ \text{Soluble.} & & & & \text{Soluble.} \end{array}$$

$$\begin{array}{ccccc} 2\times 82 & & 32 & & 2\times 98 \\ \mathrm{2H_2SO_3} & + & \mathrm{O_2} & \rightleftarrows & \mathrm{2H_2SO_4} \\ \textit{Sulphurous acid.} & & \textit{Oxygen.} & & \textit{Sulphuric acid.} \\ \text{Soluble.} & & \text{Soluble.} & & \text{Soluble.} \end{array}$$

$$\begin{array}{ccccccc} 202 & & 32 & & 136 & & 98 \\ \mathrm{Ca(HSO_3)_2} & + & \mathrm{O_2} & \rightleftarrows & \mathrm{CaSO_4} & + & \mathrm{H_2SO_4} \\ \textit{Calcium bisulphite.} & & \textit{Oxygen.} & & \textit{Calcium sulphate.} & & \textit{Sulphuric acid.} \\ \text{Soluble.} & & \text{Soluble.} & & \text{Soluble.} & & \text{Soluble.} \end{array}$$

Here again only soluble substances are formed.

317. How Chemical Precipitation Acts.—The theory of precipitation reactions is exceedingly complex. The reactions noted in Section 316 explain it only in part, and even they do not necessarily proceed as indicated. Reactions are often incomplete, and

side reactions with other substances in the sewage may take place. The structure of the different substances, however, will serve as a useful guide in the interpretation of the way in which precipitation acts.

Present knowledge of precipitation reactions enables them to be ascribed to the following properties of the precipitant, which it may possess in part or as a whole. (1) The formation by chemical reaction of insoluble or very slightly soluble precipitates which in a mechanical way enmesh suspended matters and carry them down. (2) Adsorption of dissolved or colloidal matter on the large surfaces presented by the precipitate. (3) Ionization of the precipitant to yield high-valent ions which may neutralize the electrical charges on colloidal particles and cause their aggregation and settling; positive ions precipitating negative colloids and negative ions removing positive colloids. (4) Ionization of the precipitant to yield hydrogen ions or hydroxyl ions, thus rendering the sewage more acid or more alkaline.

The floc-forming chemicals possess of themselves all of these attributes, the others possess only the last two but may give rise to the phenomena listed under (1) and (2) by causing the formation of flocs of sewage matter. The student will understand readily the two former properties. A brief explanation of the two latter, however, seems desirable.

The properties of alum for this purpose may be considered as follows. The precipitate formed is aluminum hydroxide. The solubility of this substance depends upon the hydrogen ion concentration of the liquid. At low pH values it dissolves as aluminum ion Al^{+++}, at high ones it goes into solution as aluminate ion $Al(OH)_2O^-$. Somewhere in between, it is practically insoluble. As far as floc formation is concerned therefore it is important to have the liquid at the pH of greatest insolubility. In pure water this has been found to be in the vicinity of pH 5.5. In sewage it will probably vary considerably with the nature of the sewage matters present. It has been observed, however, that the presence of high-valent positive ions causes the precipitation of negative colloids, and conversely that the presence of high-valent negative ions produces the settling of positive colloids. Hence it is seen that the best clarification is obtained not necessarily at the pH of greatest insolubility of the floc. What are the best conditions for clarification will depend upon the state of the matters to be removed. Ordinarily it may be assumed

that the mechanical and adsorptive removal of solids is brought about best by the formation of an insoluble floc, while colloidal materials are affected particularly by the presence of high-valent ions.

The importance of the hydrogen-ion concentration of the liquid is apparent, and studies to determine the optimum pH for clarification should be made always. Control of the hydrogen-ion concentration is maintained commonly by the use of sulphuric acid or lime.

318. Electrolytic Treatment.—A number of proprietary processes have been promoted in which an electrical current is passed through sewage with one or more of the following aims. (1) Production of a chemical precipitant for the removal of suspended solids; (2) neutralization at the electrodes of the electrical charges of colloidal matter and its resultant precipitation; (3) reduction and subsequent oxidation of organic matter by the nascent hydrogen and oxygen produced by electrolysis of the water; (4) deodorizing of the sewage; (5) disinfection of the sewage.

In the early installations, precipitating chemicals were formed by decomposition of the electrodes, iron hydrate being produced for example by the use of iron electrodes. In one of the later processes, known as the *direct oxidation process*, lime is added to the electrolyzed sewage. Disinfection and deodorizing are commonly laid to the production of hypochlorites from the salt contained in the sewage, or, when lime is added, to the excess hydrate alkalinity.

The consensus of opinion of leading engineers seems to be that there are no results of electrolytic treatment of the sewage which cannot be obtained more cheaply and more directly by other more common treatment processes.

SLUDGE DIGESTION

319. Sludge Digestion Defined.—Apart from gritty, mineral matter settling in grit chambers, the solids removed from sewage by sedimentation and chemical precipitation and settling to the bottom of clarification tanks accumulate in a loose, honeycombed structure, the voids of which are filled with water containing more or less dissolved solids. This accumulation of sewage solids is called *sludge*. Sewage sludge is large in bulk due to its high water content, and very putrescible due to its high organic

content. Sludge resulting from plain sedimentation contains about 95 per cent water by weight, and about two-thirds of the solids are organic in nature.

Floating solids that are removed by screens or skimming tanks and the heavy mineral solids deposited as grit in grit chambers or detritus tanks are not usually classed as sludge, but when no preliminary separation of these solids is made the sludge may contain both these elements. The term *sludge* does apply in a wider sense, however, to the deposits from the sedimentation of the effluent from trickling filters and from activated sludge aeration units.

The storage, removal and disposal of the solids deposited in clarification units is, naturally, an integral part of sedimentation and chemical precipitation processes. In a number of clarification devices, furthermore, the accumulating solids are stored for a sufficient length of time before removal from the clarification units (single-story and two-story septic tanks) or in immediate connection with them (separate sludge-digestion tanks) to permit the sludge to undergo anaerobic decomposition. This is called *septicization*, and if carried on with a view to the relative completion of the processes of decomposition, it is termed *sludge digestion.*

There are several reasons for the digestion of sewage solids, among which may be mentioned: (1) the breaking-down of the putrescible organic matter, thereby rendering the sludge inoffensive, and thus facilitating its disposal; (2) the liquefaction, gasification and compacting of the sludge incidental to the destruction of the organic solids, which result in a reduced bulk of material to be handled; (3) the physical conditioning of the sludge during digestion which increases the readiness with which the sludge is dewatered and dried; (4) utilization of the combustible gases of decomposition; and (5) employment of the digested sludge as a fertilizer in agriculture.

Sewage solids constitute a rich culture medium in which hosts of micro-organisms, notably bacteria, find an abundant food supply, providing them with energy for life, growth and reproduction. Being very complex in character, the sludge is able to support a variety of groups of organisms capable of utilizing the different types of food substances as they occur originally in the solids or as they are modified by the activities of the various organisms developing in the sludge. To a large extent the products of one group of organisms become available to other

groups, until the bulk of the nutritive elements is consumed and the sludge is rendered stable. In this final state it is said to be well digested. The end products of digestion are gases, liquids, mineral compounds and relatively simple non-digestible organic matter. Generally the latter is called "humus."

While protozoa may be seen to ingest visible particles of organic matter, most of the other unicellular micro-organisms of decomposition probably absorb their food substances through their cell membrane. To obtain nourishment from solid matter, therefore, they must peptize, or liquefy, particles that are too large to be absorbed through their cell walls. This they do by means of enzymes, catalytic[1] agents that are carried in or secreted by living cells. The correlation between the concentration of specific enzymes and the chemical nature of the substances decomposed is indeed extremely great.

In sewage treatment works, sewage solids are submitted to digestion in two different ways, (1) while remaining in contact with the flowing sewage (single-story septic tanks), or (2) after separation from the flowing sewage (two-story septic tanks and separate sludge-digestion tanks). The principles of digestion, however, are the same in both cases. Sludge digestion is an anaerobic process, and even though the flowing sewage be in immediate contact with the sludge, diffusion of oxygen into the sludge mass is so slow that after the free oxygen in the sludge has been exhausted anaerobic conditions prevail in the sludge mass.

It is a very important matter that digestion proceed (1) rapidly, (2) without interfering with the sedimentation processes, (3) without adversely affecting the nature of the flowing sewage and (4) without giving rise to offensive odors. All of these requirements can be met by appropriate design and operation of modern clarification plants.

320. Course of Sludge Digestion.—While the way in which sludge digestion operates still is understood incompletely, much has been learned about it during recent times. Three major stages may be distinguished in the course of digestion of freshly deposited sewage solids: (1) intensive acid production; (2) acid regression or acid digestion; (3) intensive digestion of more resistant materials. The various changes manifested during

[1] Catalysts are substances that promote chemical reactions without themselves entering into the reactions.

these three stages have been outlined schematically by Rudolfs[1] as follows:

DIGESTION OF FRESH SEWAGE SOLIDS
(Neither well-digested sludge nor lime being added)

I. Period of intensive acid production
   (*A*) Materials attacked
      1. Easily available carbohydrates (sugars, soluble starches, cellulose)
      2. Soluble nitrogenous compounds.
   (*B*) Organisms responsible: B. coli group, spore-forming anaerobes.
   (*C*) Characteristics
      1. Increase in acidity
      2. Solids: gray, less than half on top
      3. Odors: putrefactive, H_2S
      4. Liquid: fairly clear to slightly turbid
      5. Slight coagulation of colloidal material with heat and alcohol
      6. Disappearance of protozoa
      7. Increasing B.O.D. values.
   (*D*) Products
      1. Organic acids, H_2S
      2. Gas: comparatively large volume with high percentage of CO_2 and N_2
      3. Acid carbonates.
   (*E*) pH range: 6.8 to 5.1.
   (*F*) Results
      1. Reduction of colon organisms
      2. Retardation of proteolysis.

II. Period of acid regression or acid digestion
   (*A*) Materials attacked
      1. Organic acids
      2. Nitrogenous compounds.
   (*B*) Organisms responsible: not definitely determined.
   (*C*) Characteristics
      1. Prolongation of the low level of pH values followed by a slow rise
      2. Solids: gray to yellowish-brown, half to four-fifths at top
      3. Odors: H_2S, indol, onion (mercaptans)
      4. Liquid: slightly turbid (milky) to yellow
      5. Some to considerable coagulation of colloidal material with heat and alcohol
      6. High B.O.D. values.
   (*D*) Products
      1. Gas: small volume, with decreasing percentage of CO_2 and N_2; hydrogen formed
      2. Ammonia compounds (amines, etc.)
      3. Acid carbonates.

[1] Rept. Sewage Substation, N. J. Agr. Exp. Sta., 1926, p. 492.

(*E*) pH range: 5.1 to 6.6 or 6.8.
(*F*) Results
1. Gradual rise of pH curve
2. Acceleration of digestion
3. Foaming.

III. Period of intensive digestion of more resistant materials
(*A*) Materials attacked
1. Nitrogenous materials
(*a*) Proteins
(*b*) Amino-acids, etc.
2. Ligno-cellulose (?)
(*B*) Organisms responsible: spore-forming anaerobes, and fat-splitting organisms.
(*C*) Characteristics
1. Decreased acidity
2. Increased alkalinity
3. Slowly rising pH curve
4. Solids: dark brown to black, half to none at top
5. Odors: odor associated with methane; tarry, rubber
6. Liquid: slightly turbid to clear
7. Some to no coagulation with heat, slight to no coagulation with alcohol
8. Reappearance of protozoa
9. Rapidly decreasing B.O.D. values.
(*D*) Products
1. Ammonia and other protein degradation products
2. Organic acids
3. Gas: large volume with high percentage of CH_4, low CO_2 and N_2, and no H_2.
(*E*) pH range: 6.9 to 7.4.
(*F*) Results: sludge stable enough for disposal.

It is evident that the progress of digestion can be measured in a number of ways. Many of the tests required to establish the changes that take place are, however, quite complicated. The volume and composition of the gases produced, taken together with the pH of the digesting sludge, present probably the simplest gage of the progress of digestion as a whole and are much used. Reduction in the organic content of the sludge or increase in mineralization is another simple and valuable criterion. The variations in these three parameters are shown in Fig. 154, together with certain other characteristic changes occurring during digestion.

In sludge-digestion units all three stages of digestion are operative at one and the same time. Fresh solids are continually being added and digested solids are being removed from time to

time. This presence of material in the various stages of digestion leads under proper conditions to the establishment of a physical, chemical and biological balance such that digestion progresses rapidly and without the production of offensive conditions. Attainment of this balance should be the chief aim of the designer and operator of digestion units.

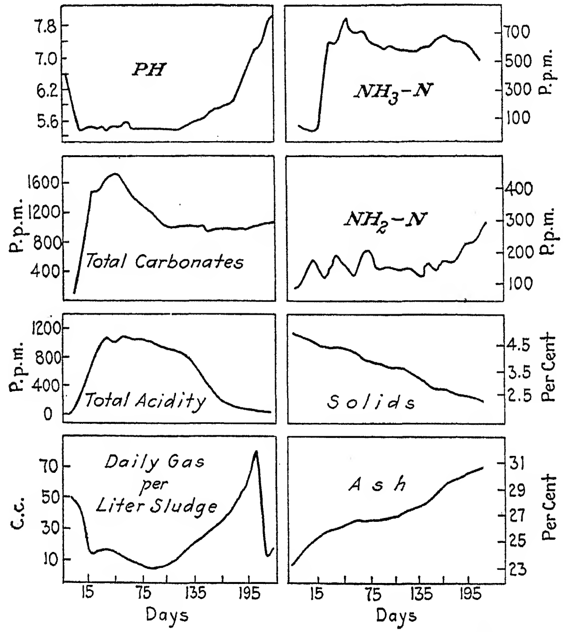

FIG. 154.—Characteristics of digesting sewage sludge.[1]

321. Factors Influencing Digestion.—Among the factors that affect digestion and are readily subject to technical control are (1) seeding, (2) temperature and (3) reaction.

Seeding.—By seeding is meant the addition to fresh solids of sludge already in process of digestion, some of it being fully digested. Seeding is important in two ways. (1) It inoculates the fresh solids with the desired organisms and enzymes of decomposition which have established themselves in the digesting material. (2) When properly controlled, it provides a balanced environment in which all organisms needful to carry the solids through the three stages of digestion find nutriment and in which reaction conditions prevail[2] which permit them to carry on their

[1] NH_2-N represents amino nitrogen.

[2] The reaction of well-digested sludge is stable and not easily altered.

life processes and thus to digest the solids expeditiously and well. The difference in the rate of digestion of seeded and unseeded sewage solids is shown in Fig. 155.

If sludge is submitted to anaerobic decomposition without seeding, as is done when a digestion tank is placed in operation after construction or after cleaning or repairs, it passes through the three stages but slowly. At the common sewage temperature of 60°F., for example, Rudolfs[1] found that about 8 months are required before the sludge can be drawn for satisfactory disposal, whereas 2 months are ordinarily adequate when the sludge is well seeded. This ratio of about 4 : 1 is observed, too, at other temperatures. The period required for balanced conditions to become established is called the *ripening* period. Once equilibrium is reached, fresh solids can be added continuously in definite quantities without destroying the balance.

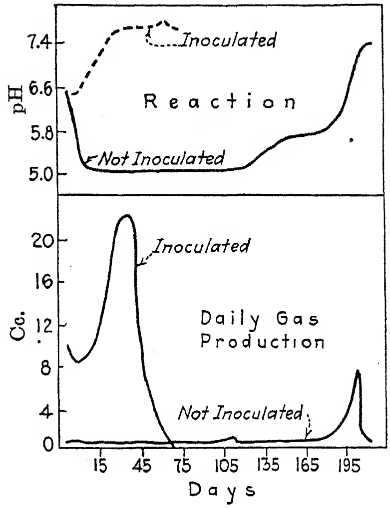

FIG. 155.—Effect of seeding upon the rate of digestion of sludge.

Rudolfs[2] found that when about 2 per cent of fresh solids by weight, on a dry basis, or about 10 per cent by volume, on a wet basis, are added daily to sludge digesting normally at about 70°F., there is an increase in the ash (mineral-matter) content of the sludge proportional to the amount of fresh solids added and that the rate of digestion is therefore presumably equal to the rate of addition of fresh material. In other words, the time required for digestion is about 50 days. The digestion schedule naturally varies with the other factors discussed in this section.

Temperature.—Since digestion is a biologically-activated process, decomposition proceeds most rapidly at temperatures at which life processes of the organisms of decay are most active. As shown in Fig. 156 the optimum conditions are encountered in

[1] *Ind. & Eng. Chem.*, 1927; **19,** 241.
[2] *Am. Jour. Pub. Health,* 1926; **16,** 365.

the vicinity of 80°F. Both above and below this temperature there is a rapid falling off in the rate of digestion. As higher temperatures (140° ±) are reached, however, digestion again proceeds more rapidly due to the activities of heat-loving (thermophilic) organisms. These higher temperatures, however, are probably of little practical significance. In sedimentation tanks in which sludge digestion is provided for, the temperature of the sludge lags only slightly behind that of the sewage. Reference should be made to Tables 67 and 68 to see that the average annual temperature of sewages in northern America is about

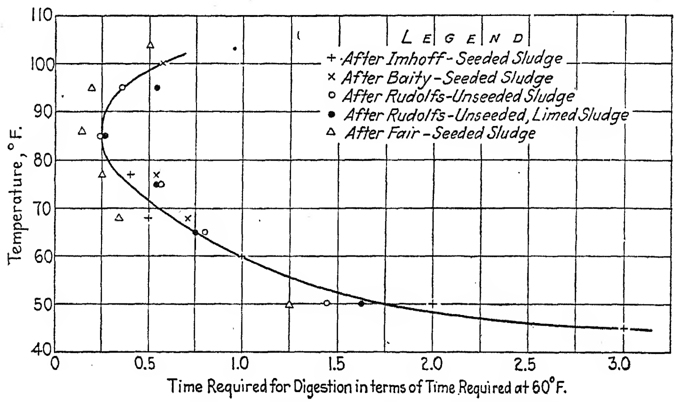

FIG. 156.—Influence of temperature on sludge digestion. Note: Additional information is required to fix more definitely the temperature-time factors here shown.

60°F. and that temperatures favoring rapid digestion are restricted to the summer and autumn of the year. Cold weather not only retards digestion, but affects the drying properties of sludge adversely. The sludge must be stored in the digestion units for longer periods of time. A knowledge of prevailing temperatures is, therefore, essential in determining upon the amount of sludge storage space to be provided in digestion tanks. Utilization of the gases of decomposition for the purpose of heating sludge-digestion tanks has recently come into vogue, as have glass-covered sludge drying beds.

Reaction.—As pointed out in Section 320, the three stages of digestion are associated with marked differences in the reaction of the sludge, the activities of the microörganisms resulting during the first stage in acid conditions which are overcome but slowly

during the second and third stages. During the period of intensive acid production, organic acids are formed faster than they are broken down; in the succeeding periods the reverse is true (see Fig. 154).

In sludge-digestion units the changes in reaction and in the accompanying rates of digestion are most pronounced (1) during the ripening period; (2) when excessive quantities of fresh sludge are added to digesting solids; (3) when large volumes of solids that have accumulated during periods of cold weather begin to "work" as the sludge is warmed by increasing temperatures; (4) when the sludge contains acid or acid-forming industrial

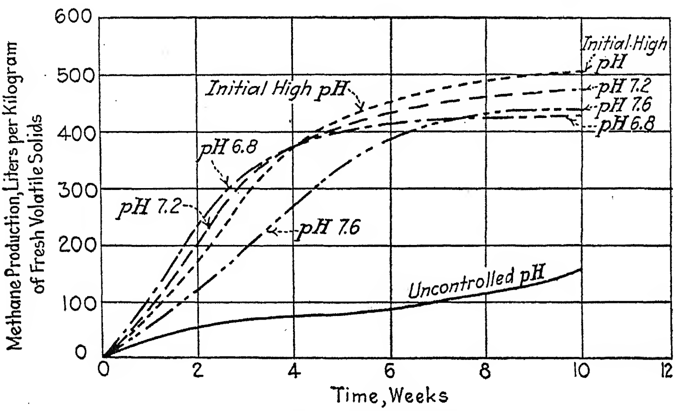

Fig. 157.—Effect of reaction control on sludge digestion.[1]

wastes; (5) when fresh sludge is allowed to accumulate or remain unseeded before passing into the digestion tank. Ordinarily in a well-designed and well-operated digestion tank it is possible, in the absence of interfering industrial wastes, to maintain a balance between fresh and digested solids such that the reaction will remain favorable. This is due to the alkaline condition of the digested material and its *buffering* action (see Section 259).

Artificial adjustment of the reaction can also be resorted to. Fair[2] has shown that the salts of alkaline-earth metals but not of alkaline metals can be used for the purpose of raising the pH of digesting sludge. Lime is most commonly employed. The effect of increasing the pH of sludge from Brockton, Mass., a sludge digestible with difficulty due to the presence of industrial

[1] Curve marked "Initial High pH" represents sludge adjusted to an initial pH of 8.0 and then allowed to take its own course. During its period of greatest methane production this sludge registered a pH of 6.8 to 7.0.

[2] *Jour.*, Bost. Soc. C. E., 1927; **14,** 82.

wastes, is illustrated in Fig. 157. The optimum pH is apparently in the vicinity of 7.2 or 7.3 but any value above 6.8 seems to produce good results.

Lime, added to digesting sludge in suitable quantities, affects digestion not only by neutralizing acid materials, but also by stimulating the growth of organisms responsible for the digestion of nitrogenous substances, and by causing the flocculation and

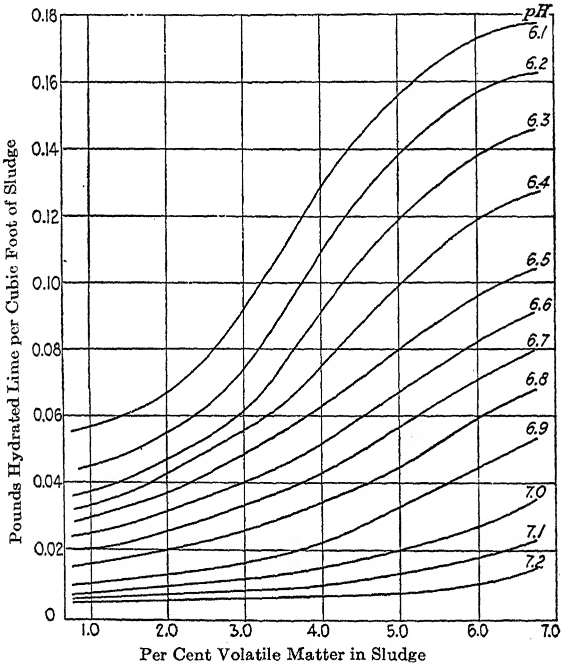

FIG. 158.—Quantity of lime required to raise the pH of fresh or partly decomposed sludge to 7.3.

precipitation of acid colloidal materials, particularly in the scum. At the same time liming permits gas bubbles held in the scum to escape, due in part probably to changes in surface tension that take place.

The quantity of lime required to raise the pH of fresh solids or partly decomposed matter to 7.3 has been determined by Rudolfs[1] to be as shown in Fig. 158.

Both activated sludge and fine screenings will digest in much the same way as fresh sewage solids. Naturally there are

[1] Rept., Sewage Substation, N. J. Agr. Exp. Sta., 1926, p. 487.

quantitative and qualitative differences, as will appear in the chapters devoted to these methods of sewage treatment.

Certain industrial wastes affect digestion adversely. Acids and alkalies may interfere. Mineral oils seem to have but little effect. Grease decomposes but slowly and may retard digestion. Small quantities of iron wastes, on the other hand, appear to accelerate the digestion processes, while increasing sedimentation and reducing odors, especially hydrogen-sulphide odors.

322. Volume Variations of Sewage Sludge.—The composition and characteristics of sewage sludge will be discussed in Chapter XIX. In the chapters preceding, however, one feature of sewage sludge will be referred to quite frequently: its variation in volume.

The volume of sludge depends mainly upon its water content and only slightly upon the character of the solid matter. The water content is commonly expressed by weight. A "90-per cent sludge," for example, contains 90 per cent of water by weight. The solid particles, in the interstices of which the water is held, vary in composition, shape and size. If one-third is composed of mineral matter with specific gravity 2.0 and two-thirds of organic matter with specific gravity 1.0, the specific gravity of the whole will be $\frac{1}{3} \times 2 + \frac{2}{3} \times 1 = 1.3$. If, furthermore, the specific gravity of the water is taken as 1.0 as it can be without appreciable error, the specific gravity of a 90-per cent sludge of this character is $0.1 \times 1.3 + 0.9 \times 1 = 1.03$. This value is a common one for average sewage sludge from strictly separate systems. If one-half of the sludge is composed of mineral matter and the other half of organic matter, computations similar to the preceding lead to a specific gravity for the solids of 1.5 and for a 90-per cent sludge of 1.05. The latter is a common value for average sewage from combined systems.

The volume of a 90-per cent sludge of specific gravity 1.03 containing 1 lb. of solid matter is $\frac{1}{8.33}\left(\frac{1}{1.3} + 9\right) = 1.17$ gal. If, however, the water content were 95 per cent the volume of the sludge would be $\frac{1}{8.33}\left(\frac{1}{1.3} + 19\right) = 2.38$ gal., or slightly more than twice as much as that of a 90-per cent sludge. If the sludge composition be considered in terms of solid matter rather than water content this fact is more apparent. A 90-per cent sludge contains 10 per cent of solid matter, whereas a 95-per cent sludge contains only 5 per cent. The volume of a 95-per cent sludge

is therefore approximately twice as great as that of a 90-per cent sludge. If V_1 and V_2 are the volumes of the sludge, P_1 and P_2 the percentages of water, and p_1 and p_2 the percentages of solid matter, we have the approximate relationships:[1]

$$\frac{V_1}{V_2} = \frac{100 - P_2}{100 - P_1} = \frac{p_2}{p_1}$$

If for example V_1 is the volume of the 90-per cent sludge and V_2 that of the 95-per cent sludge:

$$\frac{V_1}{V_2} = \frac{100 - 95}{100 - 90} = \frac{5}{10} = 0.5$$

For approximate calculations it is simple to remember that the volume varies inversely as the per cent of solid matter contained in the sludge.

FILTRATION AND OXIDATION

323. Filtration and Oxidation Defined.—Sedimentation of sewage for the purpose of separating the solids from the liquid sewage seldom reduces the putrescibility of the sewage by more than one-third as measured by its bio-chemical oxygen demand. If, therefore, sewage is to be discharged into a water course affording insufficient dilution for settled sewage, *i.e.* if the oxygen demand of the clarified sewage outweighs the oxygen supply present in the water or made available by reaeration, the sewage must be subjected to further treatment. The oxygen demand of settled sewage is exerted by organic materials that are not subject to gravitational deposition, because they are present in the sewage largely in the colloidal state and to a less extent in solution or in

[1] Let s = density of the solids
w = weight of the solids in grams
W = weight of water in grams

Then

$$V = \frac{w}{s} + W$$

$$P = 100\frac{W}{w + W}$$

$$W = \frac{Pw}{100 - P}$$

$$V = \frac{w}{s} + \frac{Pw}{100 - P}$$

$$V = \frac{w}{s}\left[\frac{100 + P(s - 1)}{100 - P}\right]$$

$$\frac{V_1}{V_2} = \frac{[100 + P_1(s - 1)](100 - P_2)}{[100 + P_2(s - 1)](100 - P_1)}$$

$$\frac{V_1}{V_2} = \frac{100 - P_2}{100 - P_1} = \frac{p_2}{p_1} \text{ approximately}$$

very fine suspension. While chemical precipitation will carry down part of this putrescible matter, experience has shown that these substances are removed or stabilized most effectively by a group of treatment processes that are variously termed "oxidation" or "biological" methods.

Oxidation may be defined as the process whereby, through the agency of living organisms in the presence of free oxygen, the organic matter is converted into a more stable form or into mineral matter. Generally speaking, oxidation of sewage is accomplished by two treatment processes, (1) filtration and (2) activated sludge.

Filtration units may take the form of (*a*) intermittent filters, (*b*) contact beds or (*c*) trickling filters. An *intermittent filter* is a natural or artificial bed of sand or other fine-grained material to which sewage is applied in intermittent doses and through which it percolates to underdrains. A *contact bed* is an artificial bed of coarse material, such as broken stone or clinkers, in a watertight basin, which is operated in cycles of filling with sewage, standing full, being emptied, and resting empty. A *trickling filter* is an artificial bed of coarse material, much the same as that used in contact beds, over which sewage is distributed intermittently and through which it trickles to underdrains.

In the *activated-sludge process*, sewage flowing through a tank is brought into intimate contact with air and biologically-active sludge previously produced by the same process. Occupying a position intermediate between filtration and activated-sludge treatment is the submerged contact aerator or Emscher filter, in which a filtering medium that is artificially aerated is suspended in flowing sewage.

324. How Changes in Sewage Are Accomplished by Filtration.—The manner in which not only suspended solids but also colloidal and dissolved materials are removed from sewage by filtration is complex. The mechanical straining-out of the larger suspended solids needs no explanation, and the factor of sedimentation which is operative in filters has been discussed fully in Section 312.

Neither straining nor sedimentation accounts for the removal of colloidal and dissolved organic matter which takes place during filtration. This action is closely associated with the development upon the filtering medium, during the ripening period of the filter, of a slimy, gelatinous growth of bacteria (zoögloea) and other

organisms such as fungi, protozoa and certain higher forms of life. It is further associated with the *contact* between the sewage and the slimy film.

At the beginning of the present century two opposing theories were advanced in explanation of the phenomena of colloid removal and oxidation of dissolved matter by filtration. The first is known as Dunbar's theory, the second as the Hampton doctrine of Travis. Generally speaking, the former stressed the importance of biological activity, while the latter gave greatest weight to physical action. There was some truth in each theory.

In the light of modern developments in the chemistry of colloids Buswell[1] explains the way in which changes are brought about in sewage by filtration on the following basis:

1. In the surface film of liquids the concentration of dissolved matter tends to change in such a way as to decrease surface tension. If, therefore, a substance dissolving in a liquid increases the surface tension of the solution, the film concentration of the substance tends to become less; if it decreases the surface tension its film concentration tends to become greater. (Most salts and all strong bases increase the surface tension of water; but ammonia and nitric and hydrochloric acids decrease it.) Substances in the colloidal state are believed to act in a similar way, and there are found in sewage colloidal soaps and proteins that tend to concentrate in a surface film. These changes in film concentration are not restricted to the air-liquid interface, but seem to apply also to the interface between the bacterial slime and the liquid. The extent of the interface or contact surface is, therefore, important.

2. At the jelly-sewage interface the following occurs:

(*a*) The substances concentrating at the interface are adsorbed to the contact surfaces and thus removed from the sewage being filtered;

(*b*) The adsorbed substances are attacked by the enzymes and living organisms present in the slime;

(*c*) As rapidly as these substances are removed by digestion or direct absorption into the living cells, others come to the interface and further removal is thus effected;

(*d*) In the presence of air the products of decomposition of organic matter are chiefly carbon dioxide, nitrates and a humus-like residue. The gas escapes from the filter; nitrates, being salts, increase the surface tension of water and therefore pass from the interface into the flowing sewage; the humus-like residue must either be removed, as in intermittent sand filters and contact beds, or sloughs off from time to time, as in trickling filters. It is an important fact that the humus from trickling filters settles readily, unlike the colloidal matter from which it largely originated.

3. Experience has shown that the oxygen present in the sewage applied to sewage filters is insufficient to maintain their oxidation reactions. The

[1] "Chemistry of Water and Sewage Treatment" Chapter XX. The Chemical Catalog Co.

filters must, therefore, be permitted to rest between doses, in order that the biological jelly may dissolve or adsorb a sufficient amount of oxygen from the air that penetrates into the interstices of the filter while it is standing idle.

To summarize, it may be said that the changes accomplished by filtration are due to surface phenomena involving (1) changes in the film concentration of the sewage at the jelly-sewage interface and adsorption of the substances concentrating at the interface, (2) the activity of living organisms and their enzymes found in the gelatinous films covering the contact surfaces, and (3) the maintenance of proper conditions by an adequate supply of oxygen. All of these changes are spoken of as *contact* phenomena.

325. How Changes in Sewage Are Accomplished by Activated Sludge.—When sewage is agitated, stirred or otherwise kept in circulation there will be formed, in the presence of dissolved oxygen, a sludge floc consisting of (1) sewage matters originally in suspension or in the colloidal state and (2) bacteria and other living organisms. This floc, as in the case of trickling filter humus, settles readily. If the concentrated sludge obtained by sedimentation is introduced into raw sewage,[1] the rapidity with which suspended and colloidal matters are coagulated may be increased greatly.

The process, as ordinarily carried out, consists of adding sludge to the sewage in proper proportion, introducing sufficient air to provide enough dissolved oxygen to maintain aerobic conditions, and agitating the mixture until practically all of the suspended and colloidal matter has been flocculated or absorbed by the floc introduced into and formed in the sewage. The mixture is then conducted to tanks where the floc is removed by sedimentation and the clear supernatant water passes away as effluent. That portion of the sludge which is not required for the treatment of the incoming sewage is diverted and disposed of.

As was the case with filtration, two opposing theories of the manner in which activated-sludge treatment of sewage acts have been advanced. One stresses the physical factors; the other, the biological factors. Generally speaking, the changes accomplished are probably closely akin to those brought about by filtration. The theory of activated-sludge treatment may be summarized as follows:

[1] The sludge thus introduced into the sewage is known as "return sludge."

The first and perhaps most noticeable function of the process is that of coagulating or flocculating the suspended and colloidal matters in the sewage. This action is similar in effect to the well-known chemical coagulation with sulphate of alumina or sulphate of iron and lime, and the floc resembles the chemical coagulum, particularly the ferric hydrate from the ferrous iron and lime treatment.

The floc is a sponge-like mass, or, as expressed by Stein,[1] "an open-mesh network" which, in the process of formation, may envelop, entrap or entrain colloidal matter and bacteria. The sponge-like structure of the floc offers a very large surface area for contact and this floc appears to be able to absorb colloidal matter, gases and coloring compounds. Buswell has estimated that the sludge surfaces present an area of 500 sq. ft. per cubic foot of tank volume. When the floc is driven about in the liquid it has a sweeping action[2] by which the colloidal substances may be said to be swept out of the water, or, as stated by Parker,[3] the "process may be regarded as passing a filter through the water in place of passing the water through a filter."

Thus far the process appears to be primarily of a physical nature. It has been demonstrated, however,[4] that it cannot be carried out under sterile conditions.

Just what the action of bacteria and other organisms may be is not fully understood. One plausible theory is that the bacteria which are contained in the cell-like structure of the floc feed upon the very finely divided matter and thus relieve the floc of its burden of such substances and restore its faculty of absorption to such an extent that, when introduced into the incoming sewage, the floc efficiently performs its function of absorbing the colloidal matter, which also will be consumed by the living organisms which thus cause regeneration of the floc. It is because of these properties that the sludge has come to be called "activated sludge," a term suggested by Ardern and Lockett.

One of the cardinal principles of this method of treatment is that there must be dissolved in the sewage an ample supply of oxygen to maintain aerobic conditions. The action of the organisms in consuming the colloidal matter is one of digestion

[1] "Water Purification Plants and Their Operation," p. 143.

[2] COPELAND, Second Annual Report, Milwaukee Sewerage Commission, Dec. 31, 1915; p. 115.

[3] "The Control of Water," p. 556.

[4] ARDERN and LOCKETT: Oxidation of Sewage without Filters. *Jour.*, Soc. Chem. Ind., **33**, 535.

or oxidation, sometimes referred to as "moist combustion." Through this action the actual weight of suspended and colloidal matter is reduced, the products of the combustion being carried off in the form of dissolved or gaseous matter. This process, under favorable conditions, may extend to nitrification, by which substantial quantities of nitrates and nitrites are formed. Nitrification, however, is not necessary to the maintenance of sludge activity.

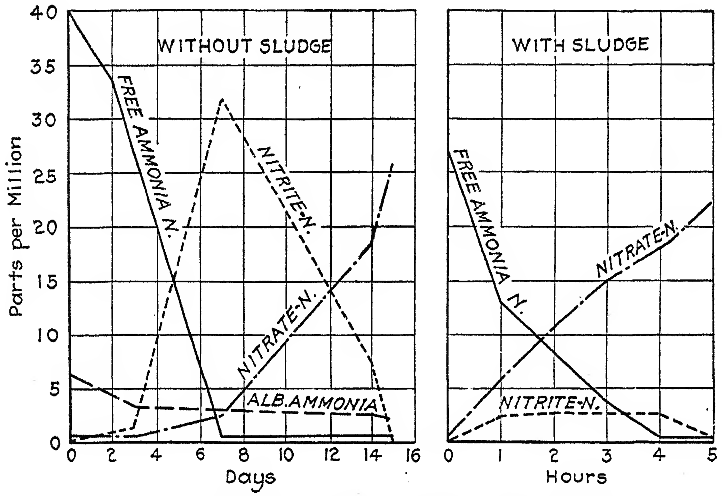

FIG. 159.—Effect of aeration with and without sludge.

The importance of the presence of activated sludge in the aerating tank is shown in Fig. 159, giving the results of parallel experiments made by Bartow and Mohlman of the University of Illinois. In one case sewage was aerated without sludge and in the other with sludge. In both cases nitrification followed the nitrogen cycle, but without sludge the completion of the cycle was a matter of days and the stages of change are distinct, while with activated sludge the cycle was completed in a few hours and the nitrite nitrogen was oxidized at once to nitrate nitrogen.

DISINFECTION

326. Disinfection Defined.—The term "disinfection" is used here to designate the treatment of sewage or contaminated water so as to reduce greatly the number of bacteria in it. Incidentally this treatment may deodorize the liquid more or less and reduce or

retard putrefaction. By "sterilization" is meant the destruction of all organisms. In the methods of treating sewage previously considered in this chapter, removal of suspended matter and change in the character of the organic matter originally present are the main objects sought, whereas in disinfection and sterilization the main object is to kill the bacteria in the liquids treated. Two of the treatment processes, however,—intermittent sand filtration and activated sludge treatment—show high bacterial removal.

It is unusual for a water receiving sewage or sewage effluents to be used for domestic consumption without previous treatment, and with modern facilities for purifying and disinfecting a water supply, it is generally wiser to apply the corrective measures to the water supply than to the sewage. There are circumstances, however, under which it is desirable to disinfect the sewage even though the water supply be effectively treated. The danger of contracting typhoid fever from polluted water at bathing beaches, and by eating sewage-contaminated oysters or other shell-fish, also justifies the disinfection of the sewage of some communities.

Disinfection may be accomplished in three general ways, by (1) heat, (2) chemicals, or (3) actinic rays (ultra-violet light). As far as municipal sewage treatment is concerned, however, chemicals are the only successful disinfecting agents, and among these chlorine and chlorine compounds offer at the present time the only practical means of securing economical and adequate results. Discussion of the theory of disinfection, therefore, hinges upon chlorination.

327. Theory of Chlorination.—In modern sewage works chlorine is commonly added to sewage as chlorine gas. Use of chlorine in this form was preceded by that of hypochlorites, particularly of calcium hypochlorite or bleaching powder. In either case hypochlorous acid, a strong oxidizing agent, is formed, as shown by the following reactions:

$$\underset{\text{Chlorine gas}}{Cl_2} + \underset{\text{Water}}{H_2O} \leftrightarrows \underset{\text{Hypochlorous acid}}{HClO} + \underset{\text{Hydrochloric acid}}{HCl}$$

$$\underset{\text{Bleaching powder}}{2CaClOCl} + \underset{\text{Carbonic acid}}{2H_2CO_3} \leftrightarrows \underset{\text{Calcium bicarbonate}}{Ca(HCO_3)_2} + \underset{\text{Hypochlorous acid}}{2HClO}$$

$$\underset{\text{Hypochlorous acid}}{2HClO} \leftrightarrows \underset{\text{Hydrochloric acid}}{2HCl} + \underset{\text{Oxygen}}{O_2}$$

Consideration of the oxidizing properties of hypochlorous acid led to the theory that destruction of bacteria by chlorine was due to the nascent, or atomic, oxygen liberated by this unstable compound. Opposed to this theory, however, were, among other things, (1) the observation that other oxidizing agents, such as hydrogen peroxide (H_2O_2) and ozone (O_3), having an oxidizing value equal to or greater than chlorine, do not possess the same disinfecting power; (2) the fact that chloramine (NH_2Cl)[1] which possesses no oxidizing power may, in certain instances, exceed chlorine and hypochlorites in disinfecting efficiency.

While the exact manner of accomplishing cell destruction by chlorine remains uncertain, death is probably due to a combination of chlorine with the cell contents.

Like all disinfecting processes the destruction of bacteria by chlorination is a time-concentration phenomenon.[2] This means that disinfection is not instantaneous, but that it depends upon the length of time during which the bacteria are exposed to the disinfectant and upon the concentration of the bacteria and the disinfecting chemicals. In the chlorination of sewage, furthermore, the unstable organic matter exerts a marked and rapid chlorine demand which must be satisfied before disinfection may be expected. The chlorine added in excess of this initial demand is called "residual chlorine." The time required for disinfection is relatively short, but must be taken into account to secure proper chlorination of sewage.

Disinfection of raw sewage is somewhat unreliable. The grosser solids are not penetrated by chlorine, and only as these matters are removed by sedimentation or other means can chlorination of sewage be given a definite efficiency rating.

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[1] $Cl_2 + NH_3 \leftrightarrows NH_2Cl + HCl$.

[2] In pure water the mortality of bacteria due to disinfecting agents is a logarithmic function of time, the number remaining after each time interval being proportionate to the number present at the beginning. Formulation of this type of relationship has been discussed in Section 265 of Chapter XI.

The following formula holds:

$$N_t = \frac{N}{K^t} \text{ or } K = \frac{1}{t} \log \frac{N}{N_t}$$

where N_t = number of bacteria living after time t

N = initial number of bacteria

t = time

K = disinfection constant

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Problems

1. A well-baffled sedimentation basin will remove 75 per cent of the particles of specific gravity 2.65 and 0.01 mm. in diameter at 50°F. What per cent of these particles will be removed at (*a*) the minimum monthly temperature, (*b*) the maximum monthly temperature of the sewage if the temperature varies as that of the sewage of Schenectady, N. Y. (1927)?

Ans. (*a*) 72 per cent.
(*b*) 82 per cent.

2. Derive a formula for the diameter of particles of specific gravity 2.00 removed in a single continuous basin in terms of the per cent of particles deposited, the c.f.s. of sewage per square foot of basin area and the temperature.

3. Assuming that the diameters of (*a*) silt and (*b*) fine sand are 0.05 mm. and 0.1 mm. respectively, what is the ratio of the bottom velocity that will move these materials to their hydraulic subsiding values? Use the data given in Table 35 and assume a specific gravity of 2.65. *Ans.* (*a*) 44:1.
(*b*) 22:1.

4. There are two grit chambers at the Worcester, Mass., treatment works. The chambers are 8 ft. wide, and have an effective length of about 75 ft. and a maximum effective depth of about 6 ft. The outfall sewer is 72 in. in diameter and has a carrying capacity of 66 m.g.d. when full. Assuming that the average flow is 28 m.g.d., determine the performance of the chambers for (*a*) average flows; (*b*) maximum flows. Note that for average flows the effective depth will not be 6 ft.

5. How much alkalinity is required to precipitate 100 lb. of alum per million gallons of sewage? *Ans.* 5.4 p.p.m.

6. How many grains of lime as CaO must be added per gallon of sewage to provide sufficient alkalinity to decompose 200 lb. of alum per million gallons of sewage? *Ans.* 0.354 g.p.g.

7. If 150 lb. of copperas are added to sewage, (*a*) how much lime should be added as CaO to permit the reaction to be completed; (*b*) how many lb. of oxygen will be used up? *Ans.* (*a*) 60.5 lb.
(*b*) 4.33 lb.

8. If 100 lb. of (*a*) alum, (*b*) copperas, (*c*) lime are added per million gallons of sewage and it is assumed that all insoluble and very slightly soluble products of the reactions, with the exception of 11 p.p.m. $CaCO_3$, are precipitated as sludge, how many lb. of sludge per million gallons of sewage will result?

Ans. (*a*) 46.8 lb.
(*b*) 38.4 lb.
(*c*) 176 lb.

9. If sewage contains 20 p.p.m. of CO_2 and 100 p.p.m. of alkalinity, all of which is due to calcium bicarbonate, how much lime is required for precipitation? *Ans.* 81.4 p.p.m.

10. How much oxygen will be used up theoretically when 700 lb. or sulphur dioxide per million gallons are added to sewage with an alkalinity of 50 p.p.m? *Ans.* 21 p.p.m.

11. Estimate in accordance with Rudolfs' rule the amount of sludge, in cubic yards, that must be present in a digestion tank receiving the settled solids from 1 m.g.d. of average sewage, (*a*) at 70°F., (*b*) at 60°F. and (*c*) at 80°F. Assume 75 per cent removal of suspended matter; sludge of 90 per cent moisture and specific gravity 1.02. *Ans.* (*a*) 548 cu. yd.
(*b*) 992 cu. yd.
(*c*) 280 cu. yd.

12. If sludge digests in 2 months at a temperature of 60°F., how long will it take to be digested at (*a*) 50°; (*b*) 70°; (*c*) 80°; (*d*) at the optimum temperature of digestion? *Ans.* (*a*) 105 days.
(*b*) 33 days.
(*c*) 17 days.
(*d*) 15 days.

13. The pH of the contents of a digestion tank is 6.8. If the tank contains (1) 3,000 cu. ft. of sludge with 8 per cent solids, of which 68 per cent are volatile, (2) 1,000 cu. ft. of scum with 6 per cent solids, of which 85 per cent are volatile, and (3) 6,000 cu. ft. of liquid with 0.5 per cent solids, of which 40 per cent are volatile, how many pounds of lime must be added to raise the pH to 7.3? *Ans.* 291 lb.

14. The water content of a sludge is reduced from 98 per cent to 95 per cent. What is the per cent reduction in volume (*a*) by the approximate method; (*b*) by the more exact method, assuming that the solids contain 70 per cent organic matter of specific gravity 1.00 and 30 per cent mineral matter of specific gravity 2.00? What is the specific gravity of (*c*) the 98-per cent sludge, (*d*) the 95-per cent sludge? *Ans.* (*a*) 60 per cent.
(*b*) 60.2 per cent.
(*c*) 1.006.
(*d*) 1.015.

CHAPTER XV

SCREENS, GRIT CHAMBERS AND SKIMMING TANKS

SCREENS

328. Objects of Screening.—Municipal sewage contains floating and suspended matters, such as cloth, paper, kitchen refuse, pieces of wood, cork, hair and fiber, and uncomminuted fecal solids, which are readily removed by screening. Among the objects accomplished by screening the following are prominent:

(*A*) In conjunction with the protection of appliances for conveying and pumping sewage and sewage sludge:
  1. Protection of pumps from injury.
  2. Protection of inverted siphons and force mains from clogging.
  3. Protection of valves and gates to insure proper operation.

(*B*) In conjunction with sewage treatment processes:
  1. Removal of floating matters that tend to form unsightly scums on settling and aeration tanks.
  2. Prevention of heavy and extremely tough floating scum on the surface of septic tanks and other sludge digestion tanks.
  3. Removal of solids likely to clog sprinkler nozzles or the surface of filters or of irrigation areas.
  4. Removal of larger solids that may settle to the bottom of aeration units where they may interfere with air diffusion or may putrefy.
  5. Removal of coarse solids and uncomminuted fecal matter that are not readily penetrated by chlorine when sewage is disinfected.
  6. As a temporary expedient for service while developing more complete methods for treatment and building the plant for applying them.

(*C*) In conjunction with sewage disposal by dilution:
  1. Removal of unsightly matters which will float on the surface of the waters receiving the sewage or become stranded on their shores
  2. Reduction in the amount of sludge settling to the bottom of slow-moving bodies of water and likely to form sludge banks or cause offense by decomposing.

329. Classification of Screening Devices.—The following definitions[1] are introduced here to avoid confusion as to the meaning of certain terms.

[1] A. S. C. E., "Manuals of Engineering Practice," No. 2, 1928.

A *screen* is a device with openings, generally of uniform size, used to retain coarse sewage solids. The screening element may consist of parallel bars, rods, or wires, grating, wire mesh, or perforated plate, and the openings may be of any shape, generally circular or rectangular slots. A screen composed of parallel bars or rods is called a *rack*. Although a rack is a screening device the use of the term *screen* should be limited to the type employing wire cloth or perforated plates. However, the function performed by a rack is called *screening*, and the material removed by it is known as *screenings*. If racks or screens are stationary they are termed *fixed*, and if they are capable of motion they are termed *movable*. According to the method of cleaning racks and screens are designated as *hand-cleaned* or *machine-cleaned*.

An important distinction is made, according to the purpose of screening, between (1) *fine screening*, which has certain elements of a treatment process, and (2) *coarse screening*, which is not primarily intended to be such a process. In a general way, the size of opening which is on the boundary between coarse and fine screening is taken by American engineers at ¼ in. By size of opening is meant the least dimension of the waterway. The relative terms used to indicate the various sizes of openings employed in racks and screens are shown in the following schedule:

	Size of Opening
Racks	
Coarse	More than 2 in.
Medium	1 to 2 in.
Fine	Less than 1 in.
Screens	
Medium	¼ in. or more. (Sizes larger than ⅜ in. are rarely used.)
Fine	Less than ¼ in.

330. Racks.—*Fixed racks* are commonly used in connection with grit chambers. It is usually best to place them at the outlet of the chambers for the following reasons: (1) When partially clogged, racks placed above the grit chambers will cause the sewage to back up in the sewers, forming deposits there rather than in the grit chambers; (2) as the sewage passes through the racks it is given an irregular, eddying motion likely to reduce the sedimentation efficiency of the chamber immediately adjacent to the racks; (3) where grit chambers are large, to retain coarse organic material, the work of cleaning the racks will be much less.

Design of Fixed Racks.—Racks and screens are generally placed at right angles in plan to the axis of the channel they cross. Their area should depend upon the quantity of sewage to be screened, the frequency of cleaning, the size of the openings and the permissible loss of head due to them. If only a narrow channel is available, the desired screening area may be obtained by curving the rack or giving it an angular shape in plan, or, more commonly, by placing it on an incline. (See Fig. 160.) Boston experience indicates that with combined sewers the rack should have a clear area at least 50 per cent greater than that of

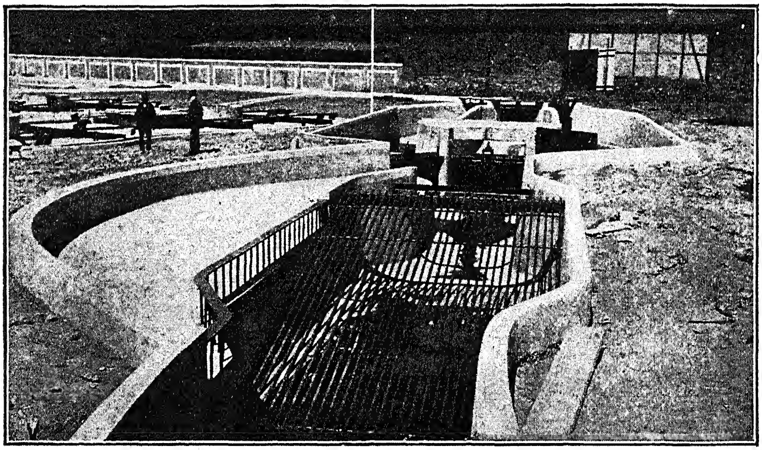


FIG. 160.—Waste-weir at rack of Emschergenossenschaft plant.

the channel leading to it. Fuller and McClintock[1] state that for domestic sewage a rack area of 20 sq. ft. per million gallons daily is ordinarily satisfactory where racks are cleaned three times during the day and are left clean at night.

The size of the openings depends upon the purpose to be accomplished by screening. Where the racks screen sewage passing to centrifugal pumps with closed impellers, it is necessary to take out material of rather small size in case the pumps are small, this size being ascertainable from tables supplied by the makers of the pumps. Where open impellers are used, the openings of the racks need not be so small, and a clear distance between bars of 5/8 in. has been found to work well. The size of the openings in racks guarding treatment works is gradually becoming

[1] "Solving Sewage Problems," p. 193.

standardized at ½ to 1½ in. A 1-in. opening has become standard in the Boston district for racks screening sewage before its disposal by dilution. Sometimes two or three racks are used in series, openings of 1¼ to 1½ in. being provided in the first rack and of ¾ in. in the last one.

Where the screening devices have to perform an important function regularly, it is necessary to install duplicate equipment so that one unit may be in service while the other is being repaired or cleaned. Duplication is provided either by using two channels, each containing one or more racks, or by placing two racks in series in the same channel. The preference is generally given to two channels because, where there is but one, repair of one unit may necessitate putting the other unit, too, out of commission.

There is considerable variation in the degree of importance attributed by managers of sewerage works to keeping the racks clean. In some small separate systems, where no storm water enters the sewers, hardly any attention is given to the removal of screenings. In some large stations, like those of the Boston Metropolitan District, the removal of the screenings is occasionally an arduous task. There is apparently no way to foresee exactly what the conditions will be, and it is the designer's duty, therefore, to make the racks and their supports strong enough so that in case they become clogged and the sewage is dammed back by them, they will not fail. If there is any likelihood of the development of a high head upon a rack, it may be desirable (1) to install a float-operated alarm system which will automatically give notice when the sewage above the rack reaches a dangerous height, or (2) to provide a bypass with a screened waste weir of sufficient length, as indicated in Fig. 160, to prevent the sewage from rising above a predetermined height. As a preliminary step in estimating the probable effect of a rack on the elevation of the sewage in the channel above it, a computation may be made by the following formula:

$$h = \frac{V^2 - v^2}{2g} \times \frac{1}{0.7}$$

where V and v represent respectively the velocity in ft. per second through the openings of the rack and in the channel above the rack, and h is the head in ft. due to the rack. The acceleration due to gravity is denoted by g, and 0.7 is a velocity coefficient suggested for this use by Frühling.

Inclined racks are more easily cleaned than vertical ones and give a larger screening area. For these reasons they are most used. The inclination is from 30 to 45 deg. from the vertical, in most cases. In detailing the construction it is desirable to curve the top of the bars backward, so that the rake used in cleaning can be pulled back freely and carry the screenings into a trough or to a platform at the top of the rack without any difficulty. Where the bars forming the rack are of considerable length and must be supported at intermediate points as well as at the ends, they should have lugs or ears by which their rear edges can be attached to the supports in such a way that these supports will offer no obstacle to the tines of the rakes used in cleaning.

The details of the sides and bottom of the channel where the rack lies should be so worked out that there are no openings larger than the clear openings of the rack, particularly where it is desired to take out material less than 1 in. in size.

In all but very small installations grooves are left in the walls of the channel for the insertion of stop planks when it is desired to repair the racks. If stop planks or their equivalent are not provided for, it will be necessary to close the channel with sand bags or some other temporary expedient in order to repair the racks.

Cleaning Fixed Racks.—No matter whether it is expected that cleaning will be infrequent or frequent, provision must be made for it and for the expeditious removal of the screenings, which have usually an offensive odor. When the racks are cleaned by hand, it is desirable to have some kind of trough or platform to receive the screenings as they are raked over the top of the racks. At a few European plants the screenings are raked over the top into a cart or small truck which is placed below the overhanging upper part of the rack, but such elaborate detailing is not common in the United States.

Mechanical cleaning is employed with some large installations of fixed racks. This is the case at Toronto, Ont., where there are six racks, two in chambers 33 ft. deep and four in chambers 14 ft. 3 in. deep. Each rack is made up of bars 10½ ft. long, ½ in. thick and spaced to give ½-in. openings. The width of a rack is 5 ft. 8½ in. They are cleaned by means of rakes attached horizontally to endless-chain belts driven by shafting and gearing at the top of the racks. The material clinging to the tines of the rakes is detached by a cleaning bar with four rows of teeth.

The bar is placed horizontally in the headframe of the mechanism for moving the chain belts and revolved so that the teeth pass between the tines of the rakes. All the refuse brought up in this way drops into a sloping tray or trough which discharges it into a screw conveyor. This conveyor also removes grit dropped into it by a bucket elevator used in cleaning the grit chamber in front of the screens. A similar device is used at Syracuse, N. Y.

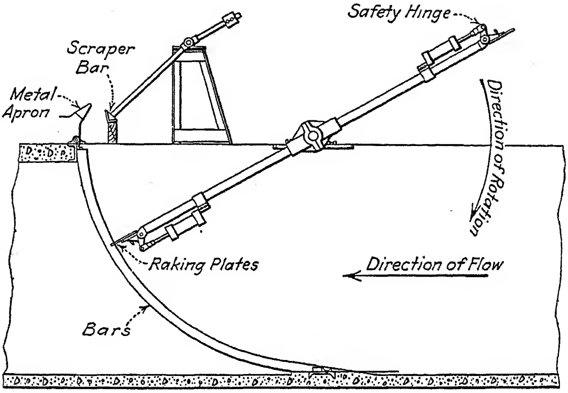


Fig. 161.—Dorrco bar screen.

In the Dorrco bar screen (Fig. 161) the bars are curved in the shape of a quadrant, concave upstream. The cleaning mechanism consists of a revolving arm carrying raking plates at each end. The plates are dentated to fit the rack and sweep the screenings up the bars. When the plates arrive at the top a scraper is engaged, which removes the solids.

Movable Racks.—There are two classes of movable racks, (1) hand-cleaned and (2) machine-cleaned. The first class predominates in the United States, the second in Europe. The only hand-cleaned movable rack of any importance is the *cage rack* shown in Figs. 162 and 163. Mechanically-cleaned racks are found in many forms, three of which are shown in Fig. 164.

Cage racks have a bottom and top of steel plates or a grid of iron bars and three sides formed of upright bars, the fourth side being the opening which faces upstream across the channel through which the sewage passes, so that the rack resembles a basket. The screens are raised vertically out of the sewage to be cleaned and must therefore be provided in duplicate, one

cage always being left in the sewage while the other is being cleaned and held in readiness for resubmersion. Fig. 162 is a plan of a typical screen chamber at Boston and Fig. 163 gives a general idea of the machinery by which a cage is operated. Each cage is about 9 ft. high and wide and 3½ ft. across from front to back. The bottom is of steel plates perforated with holes for drainage, and the sides are vertical ¾-in. bars with clear openings of 1 in. left between them. The cages are counterweighted and operated by small reversing engines. Similar cage racks more recently installed at Boston, Washington and Mil-

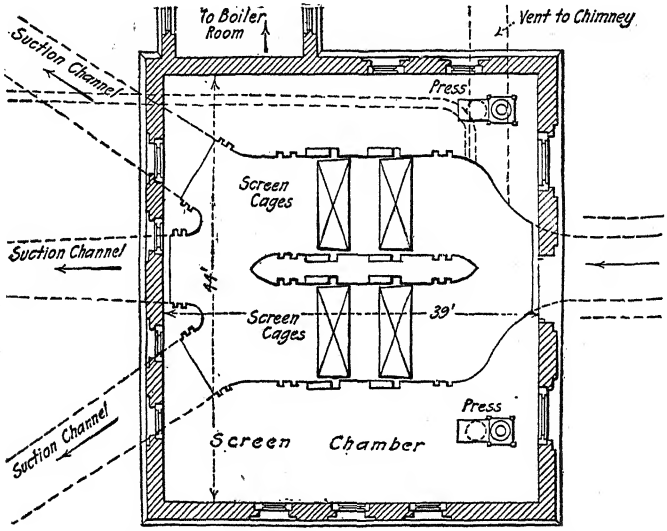
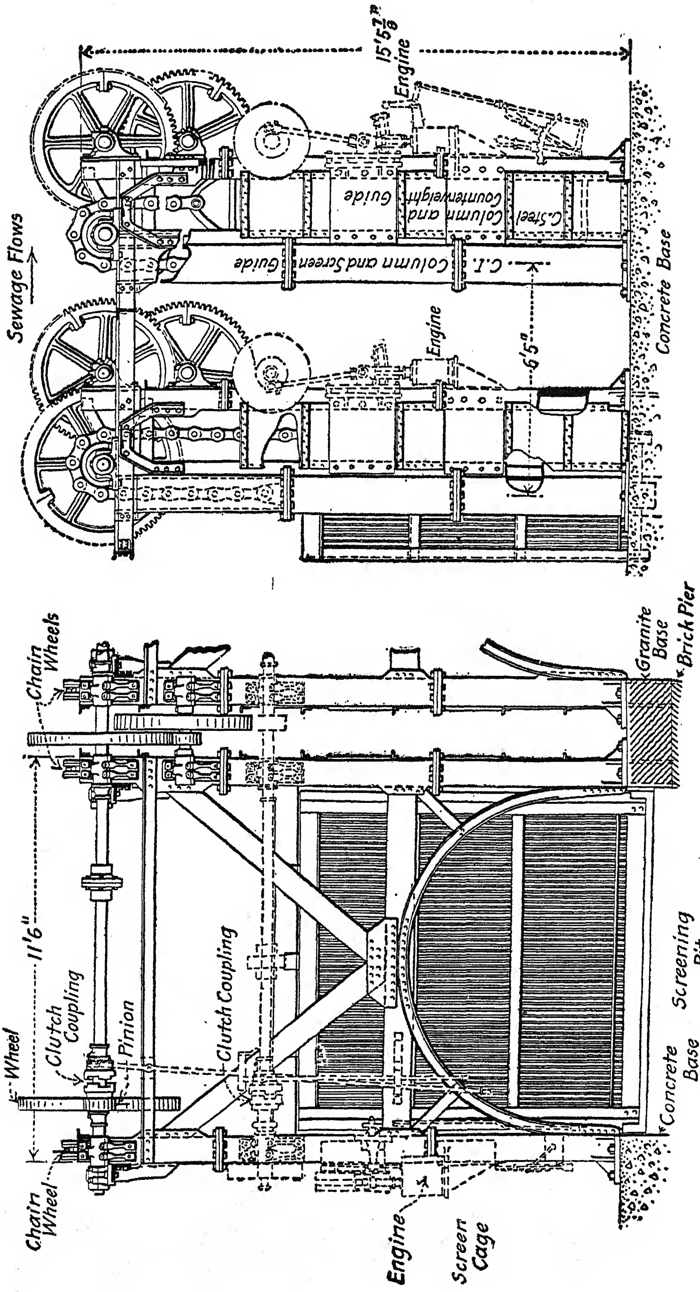


Fig. 162.—Screen chamber at Ward St. station, Boston.

waukee are equipped with electric driving motors. During heavy storms the screens need constant attendance and cleaning. At Boston the screenings are pressed in an hydraulic press to drive out a large part of the moisture and are then burned in a boiler plant in the immediate vicinity.

Machine-cleaned Racks.—Three types of machine-cleaned racks are shown in Fig. 164.

Very broad endless link belts are used as movable racks in several German cities, the most notable installation being at Hamburg. The latest of these plants there has belts made of links 14.2 in. long, held in angle-iron frames about 9.8 ft. long and 15 in. wide, so as to leave 0.4-in. openings between the links.

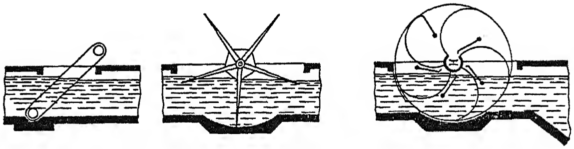


Elevation Looking Up Stream.

Side Elevation.

FIG. 163.—General features of screening machinery, Ward St. station, Boston, Mass.

Each rack consists of 46 of these frames, hinged together in such a way that the openings between the frames are very small. The links were first made of wood, then of hard rubber and finally of an aluminum alloy. The rack moves around sheaves at the top and bottom and is cleaned by a rake or stripper having a long row of rubber teeth passing across the entire width of the rack.



(*a*)-Band Rack (*b*)-Wing Rack (*c*)-Shovel-Vane Rack
FIG. 164.—Three types of European machine-cleaned racks.

The *wing* or *Frankfort rack* is a type of movable rack developed for the purpose of keeping above the sewage all parts subject to frictional wear. This is accomplished by placing three to six racks on a revolving horizontal shaft, as shown in Fig. 165. As it is necessary to have at least one rack intercepting the entire channel at all times, a depression must be made across the bottom of the channel and at least three racks used. The larger the number of racks, the smaller the depression need be.

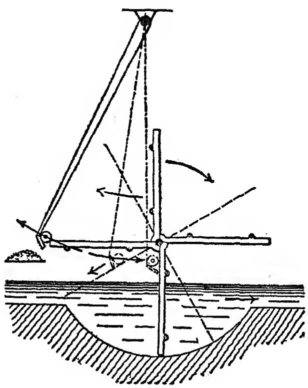


FIG. 165.—Frankfort wing rack.

The racks are cleaned mechanically. A pendulum arm hinged at the top carries a brush at the bottom, which pushes the screenings from the inner edge of each rack toward the outer edge, finally delivering them upon a belt conveyor. The face of the brush is protected by a light rake. The rack revolves in a direction opposite to the current, and the pressure tending to force material through the rack is somewhat greater than that due to the current itself. If the rack is built with small openings, the collection of screenings against its face may increase the pressure so as to make considerable power requisite for driving the apparatus.

A modification of this form of rack, in which the racks are curved instead of flat and a somewhat different method of clean-

ing them mechanically is used, has been introduced in a few European cities. It is called the *Geiger rack*, or *shovel-vane rack*.

331. Fine Screens.—Screens of perforated metal plates or wire mesh for clarifying sewage are practically all of the movable type and are either machine-cleaned or self-cleaning as will be explained later. Employed for a long time in Germany, these appliances have come into more general use in the United States during the past decade. They are intended mainly for fine screening. For this reason the screen openings are commonly smaller than $\frac{1}{8}$ in., varying from $\frac{3}{64}$ to $\frac{3}{32}$ in. in different installations. Perforated plates rather than wire cloth are generally used because of their greater resistance to wear and the greater ease of cleaning. The plates are commonly made of manganese bronze.

Three different types of screens are most used: (1) the Riensch[1]-Wurl *disk screen*, Fig. 166; (2) the Dorrco, and the Tark or Link-Belt, *drum screens*, Figs. 167 and 168; (3) the Rex, or Chain Belt, *band screen*, Fig. 169 (see Table 80). These screens have their distinctive features as to shape, operation, cleaning and handling the screenings. The Riensch-Wurl and Tark screens are cleaned by brushes. The Rex screen is cleaned by brushes, or by jets of water or compressed air. The Dorrco screen is self-cleaning. Where there is much grease in the sewage the washing of brushes with kerosene has been found useful. The condition of the screen is also improved by this treatment or by blowing steam or hot water over the screen once or twice a day.

The head lost in fine screening may be appreciable. Wurl, who developed the screen known by his name, estimated the loss of head by the ordinary orifice formula, assuming, however, that a coefficient of discharge of 0.4 would be more conservative than the values ordinarily used for water, and that from $\frac{1}{2}$ to $\frac{2}{3}$ of the submerged openings would be covered with screenings. For the Riensch-Wurl screen, he estimated that the area of submerged openings was equal to about 25 per cent of the submerged area of the screen. Hence from the formula:

$$Q = CA\sqrt{2gh}$$

$$h = \frac{1}{2g}\left(\frac{Q}{CA}\right)^2 = 9.7\left(\frac{Q}{S}\right)^2$$

[1] Pronounced Reensh.

where

Q = quantity of sewage in c.f.s.
S = submerged surface in square feet
C = coefficient of discharge, assumed as 0.4
A = effective area of openings in square feet = say, $0.4 \times 0.25S = 0.1S$
h = head lost in feet
g = 32.2 ft. per second per second

Disk Screens.—The disk, separator, or Riensch-Wurl screen provides an adequate screening area by inclining the screening surface 10 to 25 deg. from the horizontal, as shown in Fig. 166.

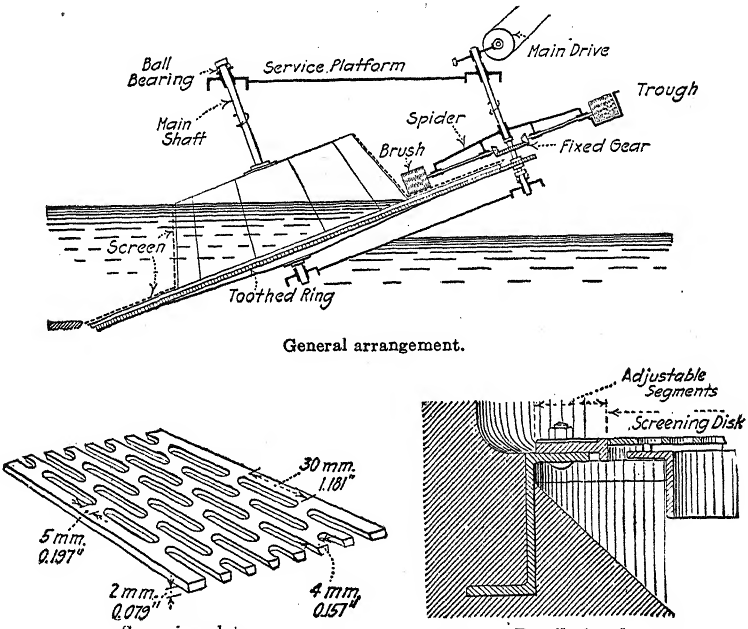


FIG. 166.—Riensch-Wurl screen.

It was developed in Germany and has been used to some extent in the United States, notably in New York City. It consists of a disk made of sheets of perforated metal, with or without a frustum of a cone attached to its center, the whole mounted on a shaft whose inclination from the vertical determines the tilting of the disk. One-fourth to one-third of the area of the disk is penetrated by the perforations through which the sewage passes.

The weight of the disks in screens up to 16 ft. diameter is now generally carried by a ball bearing at the top of the shaft. In larger sizes the shaft is stationary and carries an annular ball-bearing support on which the frame moves. In each case the bearing is above the level of the sewage.

As the screenings are raised above the surface of the sewage, they are swept off by brushes on the ends of the arms of a large spider. The brushes revolve and the arms also revolve, the combined motion of the several parts being such that every portion of screening surface of the disk is passed over at least twice. This cannot be accomplished, however, with the conical screen in the center of the disk, which is cleaned by a vertical brush of cylindrical form. At some plants the cone plates are not perforated.

Screens of this type have been constructed with disks from 4 to 26 ft. in diameter and with perforations from 0.03 to 0.2 in. wide and commonly 2 in. long. The largest plants installed in this country are those of New York City.

The characteristics of the Riensch-Wurl screens at the Irondequoit plant, Rochester, N. Y., are shown in Table 80.

Drum Screens.—The first drum screens used in the United States were of the Weand type, in which the sewage flowed outward through a conical drum covered with wire cloth. Installed at Reading, Pa., Brockton, Mass., and Baltimore, Md., they have been superseded by drum screens covered with perforated metal sheets through which the sewage passes inward.

The Dorrco screen, Fig. 167, was developed originally for screening industrial wastes. It consists of a rotating drum partly submerged in the sewage. The sewage passes into the drum from the outside and escapes through an opening at one end. The direction of rotation is such that the screen lifts the sewage within it on the side opposite the inlet so that the sewage stands from 6 to 8 in. above the level of the incoming sewage. This "head" automatically pushes the screenings off outside of the plates and they settle into a screen pit adjacent to the drum. A clean surface is thus produced, to be reimmersed in the incoming sewage. This screen is therefore self-cleaning. The screenings collected in the screen pit are dredged continuously by a bucket elevator with perforated buckets.

The characteristics of the Dorrco screens at the South Hyperion plant at Los Angeles, Cal., are shown in Table 80.

In the Tark screen, Fig. 168, the sewage passes from the outside through slotted plates into a revolving drum partly submerged in the sewage. The screened sewage escapes laterally

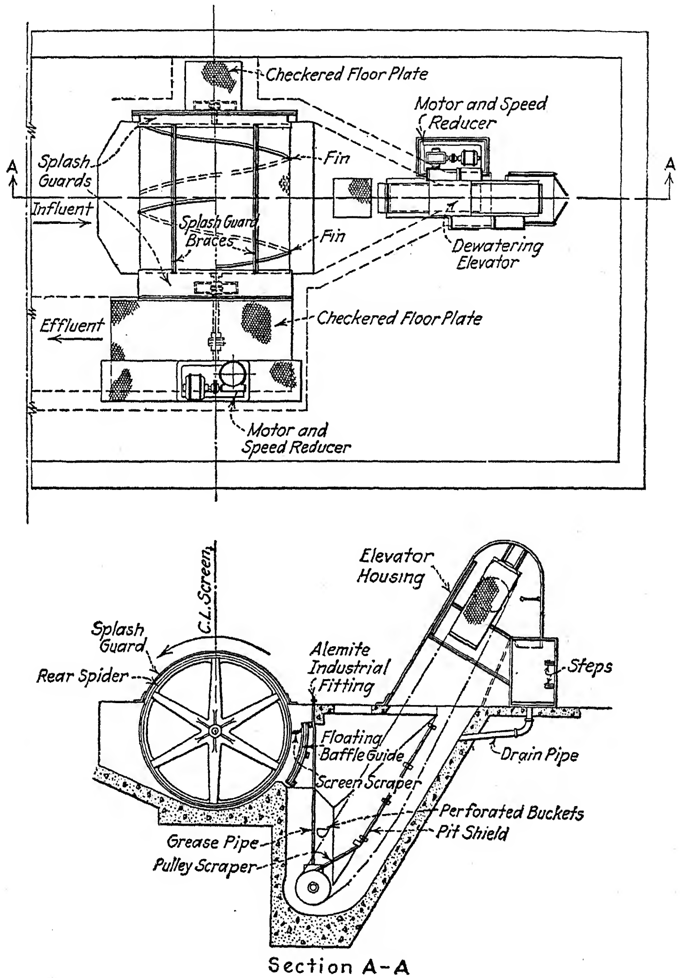


FIG. 167.—Dorrco screen

and the screenings are swept off by brushes travelling on an endless chain over the top length of the screen. The brushes are supplied automatically with kerosene or other suitable liquid to cut the grease from the screen.

Swinging plates project from the drum and scoop up any sludge that may settle in the bottom of the screening chamber. These plates are withdrawn into the inside of the drum just before they reach the brushes.

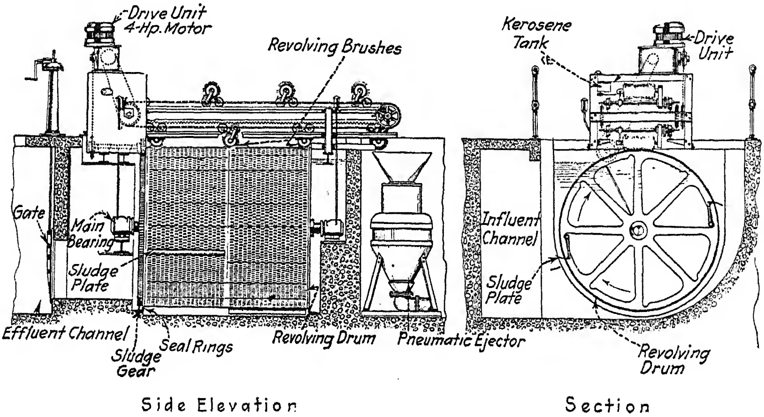


FIG. 168.—Tark screen.

The characteristics of the Tark screens at Milwaukee, Wis., are shown in Table 80.

Band Screens.—Band screens or belt screens have been used for a long time in Great Britain. In the United States their

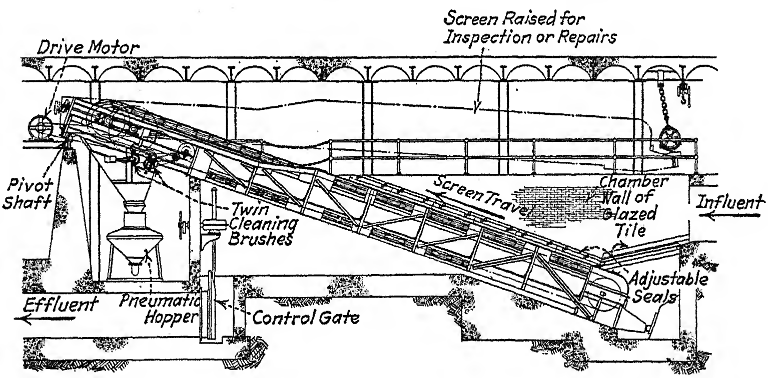


FIG. 169.—Rex screen.

use for municipal sewage is relatively new. The Rex screen, Fig. 169, consists of sections of screening plates fastened to endless chains. The sewage passes through the plates which are cleaned by jets of water or compressed air, or by brushes.

The characteristics of the Rex screens at the Canal Street plant, New York, N. Y., are shown in Table 80.

TABLE 80.—DESIGN CHARACTERISTICS OF FINE SCREENS

	Rochester, N. Y. (Irondequoit)	Los Angeles, Cal. (So. Hyperion)	Milwaukee, Wis.	New York, N. Y. (Canal St.)
Type of screen	Riensch-Wurl	Dorrco	Tark	Rex
Year installed	1916–1929	1924	1925	1925
Rated capacity, m.g.d.	120	80	150	120
Head, in.	9	3	8	
Speed: ft. per minute	.....	300	11	6–12
Speed: r.p.m.	0.7			
Number	6	8	8	3
Size				
Diameter, ft.	12	8	8	
Length, ft.	.....	8	8	55
Width, ft.	.....	.....	.....	6
Cone: Diameter, ft	6.5			
Cone: Height, ft	2.8			
Plates				
Material	Manganese bronze	Manganese bronze	Manganese bronze	Manganese bronze
Thickness, in.	$\frac{3}{16}$	$\frac{3}{16}$	$\frac{3}{16}$	$\frac{3}{16}$
Inclination, deg. from horizontal	30	.....	.....	20
Submerged area, per cent of surface	45	50	80	50
Slots				
Size, in.	$\frac{1}{8} \times 2$	$\frac{1}{16} \times 2$	$\frac{3}{32} \times 2$	$\frac{3}{64} \times 2$
Per cent of surface slotted	20.2	18.6	27.5	18.7
Operating speed: ft. per min.	.....	300	5.5–11	6–12
Operating speed: r.p.m.	0.7			
Brushes				
Number	4	.....	8	2
Speed: r.p.m.	38	.....	.....	11–23
Speed: ft. per minute	.....	.....	100	
Power consumption, k.w.h. per million gal.	3	11	2.6	3.3

332. Character and Quantity of Screenings.—In general the character and quantity of screenings depend greatly upon whether the sewers are on the separate or combined system, on the fluctuations in sewage flow, on the slopes and lengths of sewers, on the time it takes the sewage to reach the screens, on the cleanness of the interior of the sewers, on the season of the year, on the character of the population and individuals, and on any pumping to which the sewage has been subjected prior to being screened. For any particular screen they depend espe-

cially upon the size of the openings and the frequency and manner of cleaning. In recording quantities of screenings and efficiencies, it is desirable to state the dates of tests, for screens may be more effective (1) in cold than in warm weather because of less tendency of the solids to disintegration and solution at low temperatures, and (2) in the autumn than at other seasons, because of leaves washed into combined sewers.

Reliable information regarding the character and quantity of screenings is meager and somewhat incomplete. Table 81 gives a few data reported for screens of several kinds. To assist the student the following values are suggested:

Quantity of Screenings:

Racks with openings of 0.5 to 2 in.: 1 to 6 cu. ft. per million gallons.

Fine screens with openings of 3/64 to 3/32 in.: 10 to 30 cu. ft. per million gallons. 5 to 15 per cent removal of suspended matter.

Character of Screenings:

Weight per cubic foot: 40 to 60 lb.

Moisture: 80 to 90 per cent.

Volatile matter (dry basis): 80 to 90 per cent.

Whether it is practicable to ascertain, even with approximate accuracy, the screen efficiency by determining the suspended or settling solids in the sewage before and after screening is doubtful, for a very large proportion of the substances removed, especially with coarse screens, is of such a nature that it cannot be sampled fairly by practicable methods. One way of overcoming this difficulty is to measure (1) the quantity of screenings collected and (2) the quantity of suspended or settling matter in the screened sewage. Combining the two values will yield fairly reliable information.

333. Handling and Disposal of Screenings.—Where racks are cleaned by hand, the screenings are commonly taken up with broom, shovel and wheel-barrow. Where racks are machine-cleaned, screenings are either discharged directly into receptacles or conveyed to them by band or screw conveyors. Fine screens are commonly provided with receptacles into which the screenings are swept or delivered by bucket conveyors, or they are equipped with bands on which the screenings are transported to receptacles. The screenings accumulating in the receptacles are, in some cases, ejected by compressed air into carts which carry them to the point of disposal, or if screenings are disposed of in the plant or its immediate vicinity, the compressed air may convey them

TABLE 81.—QUANTITY AND CHARACTER OF SCREENINGS

	Racks					Fine screens									
	Worcester, Mass., 1926	Boston, Mass., 1926		Toronto, Can.	Plainfield, N. J., 1912	New York, N. Y., Dyckman St., 1919				Cleveland, O., W. 58th St. Testing Sta., 1916		Rochester, N. Y., 1928	New Britain, Conn., 1922	Pleasantville, N. J., 1921	New York, N. Y., Canal St., 1926
		North Metropolitan	South Metropolitan												
Type of screen	Inclined rack	Cage	Rack	Inclined rack	Inclined rack	Riensch-Wurl				Riensch-Wurl		Riensch-Wurl	Dorrco	Tark	Rex
Size of openings, in.	2.0	1.0	1.0	0.5	0.5	3/64		1/16		1/32		1/8	1/16	1/32	3/64
Time of test	...	...	...	...	...	Day	Night	Day	Night	Day	Night				
Crude sewage:															
Suspended solids, p.p.m.	...	...	...	...	...	175	48	151	55	314	150	188	153	245	226
Settling solids, p.p.m.	...	...	...	...	...	133	48	93	55						
Screenings:															
Suspended solids, p.p.m.	...	...	...	...	...	36	27	22	9	11	5	4	25	67	21
Settling solids, p.p.m.	...	...	...	...	...	36	27	22	9						
Moisture, per cent	...	...	...	...	84	78	78	82	83	87	87	92	80	...	92
Volatile (dry basis), per cent	...	...	...	...	...	88	84	91	91	...	...	...	...	...	88
Volume, cu. ft. per million gal.	1.96	1.78	4.77	2.8–3.7	5.7	27.1	9.5	18.4	9.2	...	...	6.2	18.8	32.6	
Weight, lb. per million gal.	...	...	...	125–171	261	1,449	447	1,040	447	708	306	394	1,069	...	2,180
Weight, lb. per cubic foot	...	...	...	38	46	53.4	47.0	56.5	48.6	...	...	59.8	56.9		
Efficiency of removal:															
Suspended solids, per cent	...	...	...	...	...	20.6	56.2	14.6	16.4	3.5	3.3	2.1	16.3	27.3	9.3
Settling solids, per cent	...	...	...	10–15	...	27.1	56.2	23.7	16.4						

through pipes to the point of disposal. Prior to disposal, screenings are sometimes compressed to remove some of their moisture and thus to reduce their bulk.

The offensive character of screenings makes their prompt removal desirable and their sanitary disposal imperative. The methods of disposal to be considered are (1) burying or plowing under, (2) burning or incineration, (3) composting, (4) digestion alone or with other sewage solids.

Screenings should not be buried too deeply as they will decompose but slowly if removed from the upper layers of the earth (known as the zone of "living earth") in which bacterial activity is greatest. At the same time they should be covered sufficiently well to prevent odors and fly breeding. At some plants, screenings are employed to fill low-lying areas. To avoid nuisance the screenings are covered with grit removed from the grit chambers or with earth. At Plainfield, N. J., screenings are spread upon neighboring farm lands and plowed under.

Where steam boilers are operated at the plant, screenings are often burned under the boilers after draining or compressing. Where high-temperature refuse incinerators are controlled by the municipality it is practicable to convey the drained or compressed screenings to the incineration plant for disposal. At New Rochelle, N. Y., the screenings are blown by compressed air to composting units, concrete boxes in which they are covered with 6 to 8 in. of sand or earth. After thorough "rotting" they are used as lawn dressing or for filling.

Screenings can be digested alone or with other sewage solids. Tests at Milwaukee[1] showed that for the digestion of screenings alone a schedule of 40 days could be maintained. Reduction in solids was high (61 per cent), the ash content of the raw screenings being low (9 per cent). With this digestion schedule 66.8 per cent of the organic matter, 59.5 per cent of the nitrogenous substances and 56 per cent of the fatty substances were decomposed. The digested material drained readily. The required digestion capacity of the tanks was estimated at 1 gal. per capita. Experimenting with the digestion of mixtures of screenings and activated sludge it was found that while properly seeded activated sludge digested alone at 70°F. in 30 days, 10 and 20 per cent replacement by screenings increased the digestion period to 35 and 40 days respectively.

[1] Rudolfs: *Jour.*, Boston Soc. C. E., 1929; **16**, 51.

When screenings must be removed to protect pumps, but are to be digested together with fresh sewage solids, they may be transported to the digestion units by being returned to the sewage beyond the pump discharge. If they are macerated beforehand, as at Baltimore, they may even be returned to the sewage before it is pumped, and they will settle more readily and have less tendency to form a tough troublesome scum.

GRIT CHAMBERS

334. Objects of Grit Chambers.—A *grit chamber* is an enlarged channel or long basin near the beginning of a treatment plant in which the cross-section is increased to reduce the velocity of the flowing sewage, but only enough to cause the deposition of heavy solids such as grit, sand and gravel. When the chamber is placed at the upper end of an inverted siphon or at other points of protection on combined sewers or storm-water drains it is called a *grit catcher*. The principles of design and operation of grit chambers and grit catchers, however, are the same.

The object of grit chambers is the removal by differential sedimentation of heavy mineral matter, like sand, gravel, bits of coal and cinders. In the collection and disposal of sewage, grit catchers or chambers may be desirable (1) to prevent inverted siphons and submerged outfall pipes from becoming clogged with material that often can be removed only with difficulty; (2) to prevent silt deposits in bodies of water into which sewage is discharged through outfalls or storm water overflows; (3) to prevent injury to pumping machinery.

In sewage treatment plants grit chambers are of value (1) where pumps and other mechanical devices, notably brush-cleaned fine screens, are subject to excessive wear if the grit is not removed; (2) where the removal of sludge from settling basins or digestion tanks is rendered more difficult by the presence of heavy materials that do not flow readily; (3) where the deposition of grit on digesting sludge forms a sealing layer that increases the violence of gas ebullition when the layer is lifted and breaks; and (4) where grit will weight down activated sludge and increase the difficulty of maintaining the floc in suspension or where the grit will settle on diffuser plates and interfere with aeration.

The inclusion of organic materials in grit chamber deposits should be avoided as far as possible, because they render the grit offensive and make its disposal more difficult.

335. Character and Quantity of Grit.—In sanitary sewage, grit may originate in the wash water from bathrooms and kitchens; from the washings of floors and cellars; from silt which enters the sewers through openings and loose joints; from illicit groundwater or storm-water connections; and from manufacturing processes. In storm water, grit may come from the washings of streets, roofs and yard areas; from silt entering the drains through openings and loose joints and from the draining of excavations. In combined sewage all these sources may contribute grit, and there is naturally more grit in combined sewage than in strictly separate sewage. Catch basins installed on combined systems do not obviate the necessity for grit chambers, but do reduce the load placed on them. Grit chambers usually are omitted when the sewerage system is constructed and maintained on the separate plan.

The mineral constituents of grit as removed from the chambers resemble wet sand and gravel; the organic solids are chiefly raisin and other fruit seeds, coffee and tea grounds, rags, paper, bits of meat and vegetables, and in the United States of recent years, grain. As determined by the loss-on-ignition test, the organic matter may vary from 10 to 50 per cent. The higher values are commonly associated, however, with too low velocities. The quantity allowable depends upon the location of the chamber and the disposal area. At Cleveland it was found that grit with 15 per cent volatile matter was inoffensive and could be satisfactorily disposed of in a residential district by filling low land. The specific gravity of grit is about 2.0 and it contains about 35 per cent void space. Its weight is therefore about 85 lb. per cubic foot when dry, and slightly more than 100 lb. per cubic foot when wet.

The quantity of grit is usually expressed in cubic feet, cubic yards or pounds per million gallons of sewage. The quantity of grit deposited depends not only upon the design and operation of the chamber, but also upon a number of collecting-system factors. Among the latter are the following: (1) separation of storm water and sewage; (2) topography and surface of the sewered area, in particular the nature of street and yard paving; (3) capacity of intercepters as compared with dry weather and storm flows; (4) sewer grades; (5) efficiency of catch basins and their cleaning.

With a velocity of 1 ft. per second and a detention period of 1 min., the average amount of grit deposited is commonly from

2 to 3 cu. ft. per million gallons. These values, while of service in deciding upon the quantity of grit to be handled and disposed of, cannot be used in fixing the storage capacity of grit chambers. For the latter, the quantity of grit deposited during storms between cleanings commonly controls. To allow for maximum storm demands between cleaning operations, grit storage of 10 to 30 cu. ft. per million gallons generally is provided, depending upon many conditions. With these values, periods between cleanings may drop from an annual average of two weeks to only a few days at times of successive storms. In northern climates a winter suspension of cleaning up to 6 weeks is common. Where the grit is removed continuously the chambers and equipment must be able to cope with the grit load caused by the maximum storm.

336. Design of Grit Chambers.—The factors to be considered in grit chamber design are: (1) velocity of flow and methods of obtaining uniform velocities with varying rates of sewage flow; (2) detention period; (3) grit storage; (4) method of cleaning; (5) shape of chamber.

Experience has shown that a velocity of 1 ft. per second will permit the deposition of the bulk of the heavier mineral solids without including so much organic matter as to render the grit offensive. A range of 0.5 to 1.0 ft. per second generally is employed. The velocity fixes the cross-sectional area of the chambers. In order to keep the velocity within the desired limits it is usually necessary to provide two or more independent chambers to allow for fluctuations in sewage flow. In some cases the velocity is controlled by varying the width of the chamber at different depths. A separate channel for the dry-weather flow is frequently considered desirable. Influent and effluent conduits should be so designed that the sewage enters and leaves the chambers with a minimum of turbulence while maintaining self-cleaning velocities.

The period of detention commonly employed is 1 min. This fixes the length of the chambers, and since the ratio of length to depth must not be too great (Section 313), the effective depth is delimited at the same time. Since sedimentation of granular solids is dependent to a large extent upon the surface area of the chambers, their width should be made as great as possible, while remaining consistent with other requirements. The separate units must not be made too wide for convenient cleaning, nor for the maintenance of a uniform velocity in the cross-section.

Computations based upon Hazen's theory of sedimentation will aid in the selection of proper widths as well as of the other dimensions.

The quantity of grit to be stored has been discussed in Section 335. Storage space is provided either throughout the length of the chambers or by means of one or more pits deeper than the remainder of the basins. In allowing for grit storage, some

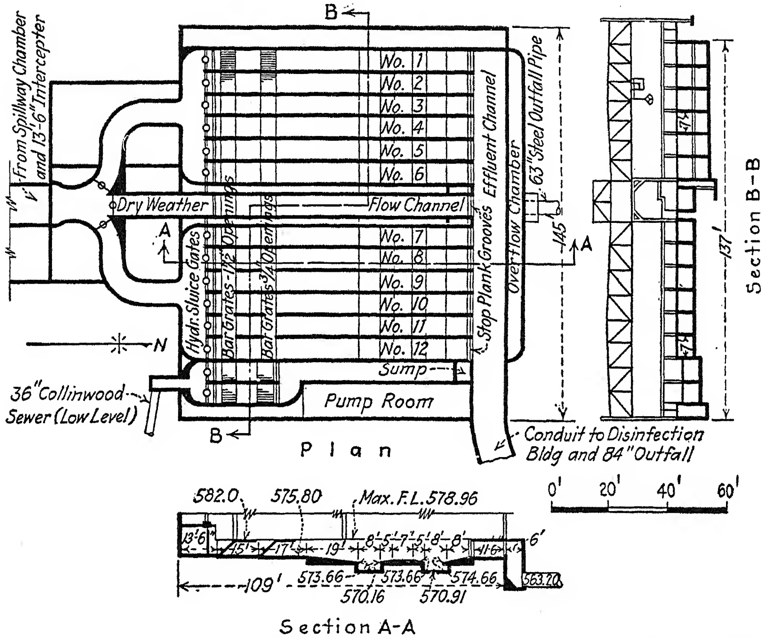


Fig. 170.—Grit chambers at Easterly Sewage Works, Cleveland, Ohio. (*After Gascoigne.*)

uncertainty is introduced in estimating the velocity of flow through the chambers. Experiments have indicated the frequent existence at, or near, the invert levels of the influent conduits of a plane below which stagnation occurs. In estimating velocities, some movement below this level should be provided for, if the velocity of flow through the chamber is to be high enough to prevent the settling of organic solids when the grit storage space is clean. Concentration of grit storage in pits may be useful in this connection as well as for cleaning purposes. Reduction of the cross-section below the invert elevation and the separation of

the flowing-through section from the grit collecting section by means similar to those employed in Imhoff tanks (Section 348) have been resorted to also.

The methods of grit storage are commonly predicated upon the processes of cleaning to be employed, which will be described in the next section of this chapter. As previously indicated other factors also must be considered. Methods of grit storage together with velocity factors will determine the shape of the chambers.

The grit chambers at the Easterly sewage treatment works at Cleveland, O. (Fig. 170), will serve to illustrate the design of grit chambers. The characteristics of these chambers, as described by Gascoigne,[1] are as follows:

Number of units: 12 plus 1 dry-weather channel

Dimensions of units: length 60 ft.; width 7 ft.; chamber depth (under conditions of maximum flow) 3.2 to 5.55 ft.; pit depth 6.05 to 8.84 ft.

Population served: 575,000.

Average daily flow (1923–1925): 83.6 million gal.

Average dry-weather flow (design value): 92 m.g.d.

Capacity of each chamber: 12 to 24 m.g.d.

Maximum-flow capacity of plant: 288 m.g.d. = 3.13 × dry-weather flow.

Design velocities: 0.5 to 1.0 ft. per second.

Detention period: 60 to 120 sec.

Grit storage space: 13 cu. yd. per pair of hoppers = 156 cu. yd. in all 12 chambers. Additional space below flow line of outlet conduit, 243 cu. yd. Available capacity for maximum flow, 1.38 cu. yd. per million gallons, and with 25 per cent ineffective, 1 cu. yd.

Frequency of cleaning (1923–1925): 5 to 36 days, average 16 days.

Method of cleaning: electrically-operated grab bucket traveling on a monorail system, loading into industrial cars.

Grit removed (1923–1925): average, 2.1 cu. ft. per million gallons; maximum, 7.3 cu. ft. per million gallons.

Composition of grit: 18 per cent volatile matter.

Racks: two bar racks, the first with 1½-in. openings, the second with ¾-in. openings.

Cost: $1,245 per million gallons for grit chambers and essential appurtenances.

337. Cleaning Grit Chambers.—There are a number of ways of cleaning grit chambers, the choice of cleaning devices depending largely upon the size of the plant and the frequency of cleaning required. The methods may be classed as (1) hand cleaning; (2) mechanical cleaning; and (3) hydraulic cleaning.

[1] *Trans.*, A. S. C. E., 1927; **91**, 496.

Hand cleaning is commonly employed at small plants, the grit being shoveled into containers varying in size from buckets to industrial railway cars. The containers are lifted out of the chambers by hand hoists or by power-driven stationary or

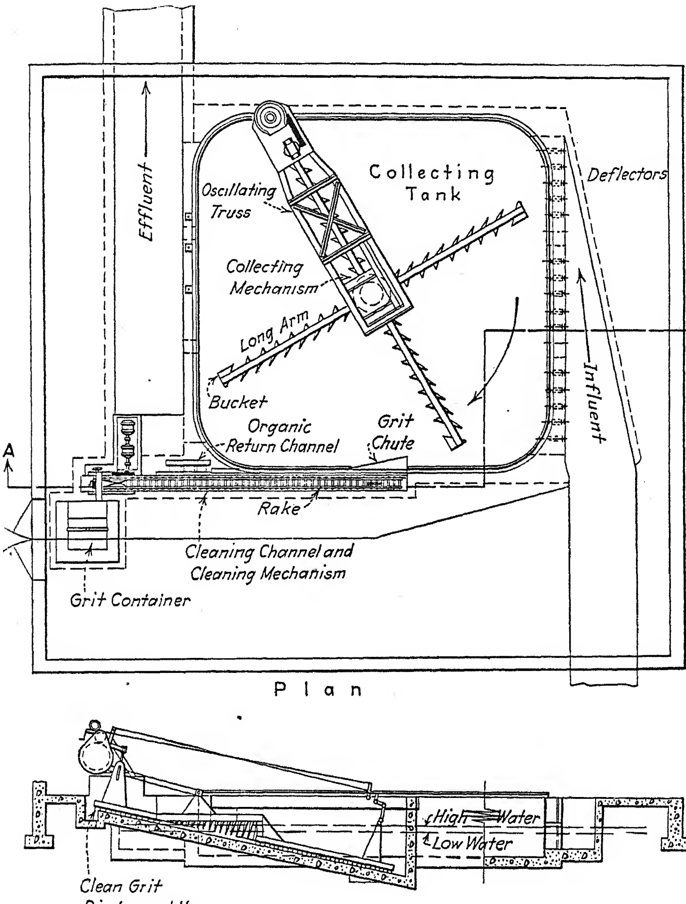


FIG. 171.—Dorr detritor.

movable cranes. Hand cleaning requires the unwatering of the chambers. This is commonly accomplished by fixed or portable pumps such as diaphragm or centrifugal pumps.

Mechanical cleaning devices take the form of bucket elevators, scrapers and clamshell buckets on mounts of various types.

These permit cleaning without unwatering and during operation; although chambers often are both unwatered and bypassed during mechanical cleaning where this is not strictly necessary. Where basins are cleaned by clamshell buckets, it is desirable to prevent injury to the floors by protecting them with some hard covering, such as granite block pavement or steel plates. Continuous cleaning by mechanical devices, notably scrapers, is coming into use.

The "Dorr detritor" shown in Fig. 171 is designed as a square basin with rounded corners and equipped with a grit-collecting and removal mechanism. The principle of operation of the collecting mechanism is similar to that employed in sedimentation tanks for the removal of sewage solids (Section 360). Movement of the grit, however, is outward rather than inward. Plows mounted on four radial revolving arms carry the grit to an inclined channel along one side of the basin. A series of reciprocating rakes move the grit up the inclined floor to the discharge point at the upper end. While passing up the incline organic solids are separated from the grit and carried back into the basin through a special opening. A cleaner, drier grit is said to be obtained by this method than in the usual way.

Hydraulic devices worthy of note are eductors, sand dredges and monitors. The first two devices lift the grit out of the chambers as a watery *slush*. Eductors are commonly connected to pits while sand dredges may travel the length of the chambers. Monitors are fire nozzles that direct a stream of water from some central point into the chamber and flush the deposits out through pipes in the sidewalls or bottom of the chamber. They are particularly applicable to side-hill locations.

338. Disposal of Grit.—Grit should be disposed of by burial, unless it contains relatively small quantities of organic matter. It has been mentioned that at Cleveland, O., the grit contains about 15 per cent volatile matter and is used for filling low-lying areas in a residential section. Filling is probably the most general method of disposal, and it is possible sometimes to use grit for covering screenings which are to be disposed of at the same time. Sometimes clean grit is used as a top dressing for dirt or gravel paths and drives used for light or occasional traffic. It is said to be of value,[1] too, for mixing with soil in the raising of certain garden crops, notably cucumbers, squashes, melons and tomatoes.

[1] SKINNER: *Trans.*, A. S. C. E., 1927; **91**, 532.

Washing the grit for the removal of offensive organic material has been undertaken, sometimes with indifferent success.

SKIMMING TANKS

339. Objects of Skimming Tanks.—A *skimming tank* is a chamber so arranged that floating matter rises and remains on the surface of the sewage until removed, while the liquid flows out continuously under partitions, curtain walls or deep scum boards.

The object of skimming tanks is the separation from the sewage by flotation of the lighter, floating solids. The material collecting on the surface of skimming tanks, whence it can be removed, includes oil, grease, soap, pieces of cork and wood, vegetable debris and fruit skins, originating in household and industry. The removal of these substances, notably oil and grease, is desirable (1) where the formation of unsightly scum or "slick" on waters receiving otherwise untreated sewage is to be avoided; (2) where similar conditions are to be obviated in the tanks of sewage treatment works; (3) where heavy discharges of oil and grease would interfere with aeration in the activated sludge process of sewage treatment (see Chapter XVIII) or with the reaeration of sewage-polluted waters.

The use of skimming tanks for the treatment of municipal sewage is a relatively new development brought about by the increased use of oil for domestic and industrial purposes and the discharge of waste oil and grease into the public sewers from garages. In non-manufacturing communities the ether-soluble matter is commonly estimated at 20 gm. per capita daily, or over 8 tons per 1,000 population per year. This is in itself a considerable quantity, and is greatly increased by sudden discharges of oily and greasy wastes in relatively large quantities that do not appear in average analyses. Not all of this material, however, will rise to the surface of skimming tanks. Much of it is found as a coating on heavier particles and escapes with them.

340. Design of Skimming Tanks.—The design of skimming tanks dealing with municipal sewage has not yet crystallized. The few American tanks in existence or under construction are elliptical or circular in shape and provide for a detention period of 1 to 15 min. The outlet, which is submerged, is situated opposite the inlet and at a lower elevation, to assist in flotation

and to remove any solids that may settle. The characteristics of three American tanks are listed in Table 82.

TABLE 82.—DESIGN CHARACTERISTICS OF SKIMMING TANKS

	Washington, D. C.	Toledo, O.	Akron, O.
Quantity of sewage treated, m.g.d.			
Average	85	21	33
Maximum	....	101	94
Average detention period, min.	1	1.8	12.6
Number of tanks	1	1	2
Shape	Egg-shaped	Elliptical	Circular
Length, ft.	22.75	21	55 (diameter)
Width, ft.	16	13	
Maximum depth, ft.	19	23	12
Inlets*			
Size, ft.	5 (diameter)	6 (diameter)	4 × 3
Depth below flow line, ft.	6	12	5
Outlets			
Size, ft.	5 (diameter)	5 (diameter)	3.5 × 3.5
Depth below flow line, ft.	16	23	8

The skimming tanks designed by the authors for Akron, O., serve a two-fold purpose which is described in Section 342.

A German skimming tank designed by Kremer is circular in shape with a central downward-directed inlet much like a Dortmund tank (see Section 345). The tank is provided with a circular baffle under which the sewage must pass to rise near the walls and to flow over a circular weir into the outlet conduit attached to the walls. The light materials to be separated from the sewage rise to the surface in the inner compartment created by the baffle. A weir and outlet conduit for the grease and oil are attached to the baffle. This weir is placed at a higher elevation than the sewage weir and thus permits only the materials lighter than water to overflow. Imhoff has advocated the use of long trough-shaped tanks with a detention period of 3 min., air being blown into the sewage from the bottom of the tank to separate the oil and grease from the remaining sewage matters to which they may be attached.

341. Removal and Disposal of Floating Matters.—The materials collecting on the surface of skimming tanks are com-

monly removed by hand. They are either dipped up or are collected on perforated trays that are dragged through the sewage until covered with floating materials. The scum may be driven to collecting points by revolving arms or by jets of water directed almost horizontally over the water surface. If the material has been dipped up or otherwise contains much water, it is sometimes placed in separating tanks from which the water is drawn off, or it is discharged onto drying beds similar to sludge-drying beds (see Section 432) which are sometimes covered with a layer of sawdust. The drained material may be buried, burned, or mixed with dried sludge from settling tanks and used for filling.

Very little information is obtainable on the quantity of floating matter that readily can be removed from sewage. At Washington, D. C., about 100 lb. per day are obtained from 85 m.g. of sewage; at Worcester, Mass., about 100 cu. ft. of oil and grease per 1,000 population were skimmed from the surface of Imhoff tanks during 1926.

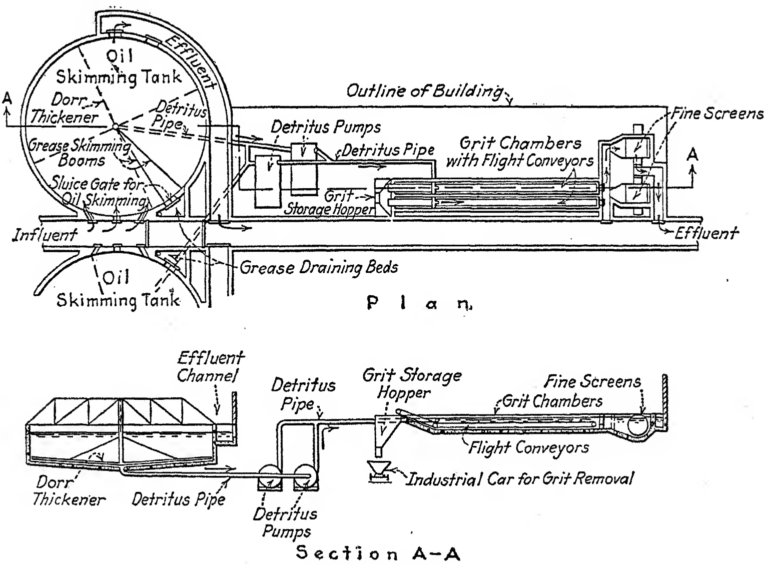


FIG. 172.—Diagrammatic sketch of skimming-detritus tanks at Akron, Ohio.

342. Skimming Detritus Tanks.—In the tanks designed for Akron, O., separation of the heavy, readily settling solids and of the light floating materials is effected in a single unit called a *skimming detritus tank* (see Fig. 172). The characteristics of

this tank have been given in part in Section 340. Floating wooden booms are provided to skim the grease from the surface of the tank and to concentrate it in front of an adjustable weir over which it is pushed onto a draining bed. A mechanism carrying revolving plows is provided to move the grit and other solids settling from the sewage to a central hopper, whence it is pumped with about 20 per cent of the average sewage flow to grit chambers in which the real separation of grit takes place. After fine screening this so-called *underflow* passes into the main conduit of the plant carrying the principal flow of sewage that passes directly through the skimming detritus tanks.

343. Grease Traps.—Grease traps are in reality small skimming tanks connected with the house or industrial sewer. Situated so close to the source of grease, which is often discharged with hot water, it is essential that they be given sufficient capacity to permit the sewage to cool and the grease to separate after congealing. Grease traps are employed at many manufacturing plants, garages, hospitals and hotels. A number of proprietary tank patterns are in use. In most of them the inlet is situated below the surface and the outlet at the bottom. To be efficient they must be large enough to hold and, if necessary, cool the sudden discharges of oily or greasy wastes and they must be cleaned frequently and regularly. Neither of these requirements is met in many cases. In the army cantonments constructed during the Great War, a capacity of 300 gal. was found desirable to serve a 250-man kitchen.

Literature

Allen, Kenneth. The Clarification of Sewage by Fine Screens. *Trans.* A. S. C. E., 1915; **78,** 880.

Gascoigne, G. B. Grit Chambers for Sewage Treatment Works. *Trans.*, A. S. C. E., 1927; **91,** 496.

Problems

1. Two inclined racks with 1½-in. openings in adjacent channels are to screen the sewage from a combined system arriving at the treatment plant in a circular sewer; $d = 5$ ft. 0 in.; $n = 0.013$; $S = 0.00087$; maximum carrying capacity three times the dry-weather flow. Find (*a*) the area of the racks and their inclination, (*b*) the size of the steel bars composing the racks, (*c*) the minimum and maximum loss of head through the racks that you would expect; (*d*) make a sketch of one of the steel bars showing its thickness, length, width and method of support; (*e*) estimate the amount of screenings to be handled.

2. If in Problem 1 the inclined racks were to be replaced by cage racks with steel rods with 1-in. clear openings what would be the answers to (*a*), (*b*), (*c*) and (*e*)?

3. At Frankfort, Germany, three wing racks possessing five vanes are used. The racks are 19.6 ft. in diameter and screen from 10 to 13 m.g.d. of sewage each. The effective radial length is 8.2 ft. Each vane consists of 0.16 by 1.20-in. steel bars, 0.4 in. apart. The ratio of the free openings to the entire vane is 1:2.4. If the sewage flows in the approach channels at a velocity of 1 ft. per second, (*a*) what must be the width of the racks; (*b*) what must be the depth of the depression in the channel; (*c*) what must be the elevation of the shaft above the channel? Illustrate with sketch.

4. The link bar rack at the main outfall works of Hamburg, Germany, has been described in Section 330. There are two racks, each 10.8 ft. wide and 19 ft. high over all. Their inclination to the horizontal is about 63 deg. The screens are subjected to a tidal range of 6.2 ft. above normal flow. The ordinary depth on the racks is 6.0 ft. The upward velocity of the racks is 1 to 2 in. per second. The usual head is 8 in., but the head builds up to 30 in. at times due to collected screenings. The sewage flow varies from 140 to 700 c.f.s. (*a*) Find the maximum thrust on the two shafts. (*b*) Assuming ratio of free openings to total screening surface as 1:1.5, what is the coefficient of discharge for worst conditions?

5. Measurements of the loss of head through Riensch-Wurl screens are found to be as follows: size of openings, ⅛ in. × 2 in.; flow, 25 m.g.d.; loss of head, 1.2 ft. The screens are 12 ft. in diameter and are inclined 30 deg. from the horizontal. The total area is approximately 127 sq. ft., 45 per cent of the surface is submerged when the screens are clean, and 20.2 per cent of the total area is slotted. By Wurl's method of analysis (*a*) what is the coefficient of discharge for clean screens? (*b*) Assuming a coefficient of discharge of 0.5 and that 25 per cent of the openings are covered with screenings, what is the area of openings submerged?

Ans. (*a*) 0.38.
(*b*) 11.7 sq. ft.

6. What is the maximum self-cleaning force in lb. per square inch exerted on a clogged opening in a Dorrco screen?

Ans. 0.29 lb. per square inch.

7. Estimate the daily volume (*a*) and weight (*b*) of screenings collected on a fine screen handling 40 m.g.d. of sewage. (*c*) How many gallons of water will these screenings contain and (*d*) how much volatile matter? If these screenings are disposed of by burial in 36-in. deep trenches with 4-in. cover and 1 year is required for decomposition, (*e*) what acreage of disposal area is needed?

8. Make an analysis of the grit chambers at the Easterly sewage treatment works of Cleveland, O., on the basis of Hazen's theory of sedimentation. Determine the performance of the individual chambers for flows of (*a*) 12 m.g.d.; (*b*) 24 m.g.d.

9. Determine the annual quantity of grit in cu. yd. to be disposed of by the Easterly sewage treatment works of Cleveland, O., on the basis of the performance observed for 1923–1925. *Ans.* 2,370 cu. yd.

10. An analysis of the grit collected at the Irondequoit plant of the Rochester sewerage system showed that 1 cu. ft. drained and packed weighed 85 lb. (*a*) Estimate the specific gravity of the material assuming 35 per cent void space. (*b*) A sample of the grit showed 23.3 per cent loss on ignition. If the specific gravity of the organic matter were 1.2, what would be the specific gravity of the mineral constituents? *Ans.* (*a*) 2.1.
(*b*) 2.4.

11. A grit chamber installation is to be designed for a city with an average dry-weather flow of 1 m.g.d. and a storm flow equal to 3 times the dry-weather flow. What values would you choose for the following: (*a*) minimum detention period; (*b*) maximum rate of flow; (*c*) length of units; (*d*) width of units; (*e*) depth of units; (*f*) number of units; (*g*) grit-storage capacity; (*h*) average number of days between cleanings? Give the reasons for your choice. Make a sketch of the plant.

12. What are the velocities of flow at the maximum cross-section of the skimming tanks at (*a*) Washington, D. C., and (*b*) Toledo, O.?
Ans. (*a*) 0.38 ft. per second.
(*b*) 0.19 ft. per second.

13. Make a sketch of the Kremer skimming tank described in Section 340. What objections do you see to this type of tank?

CHAPTER XVI

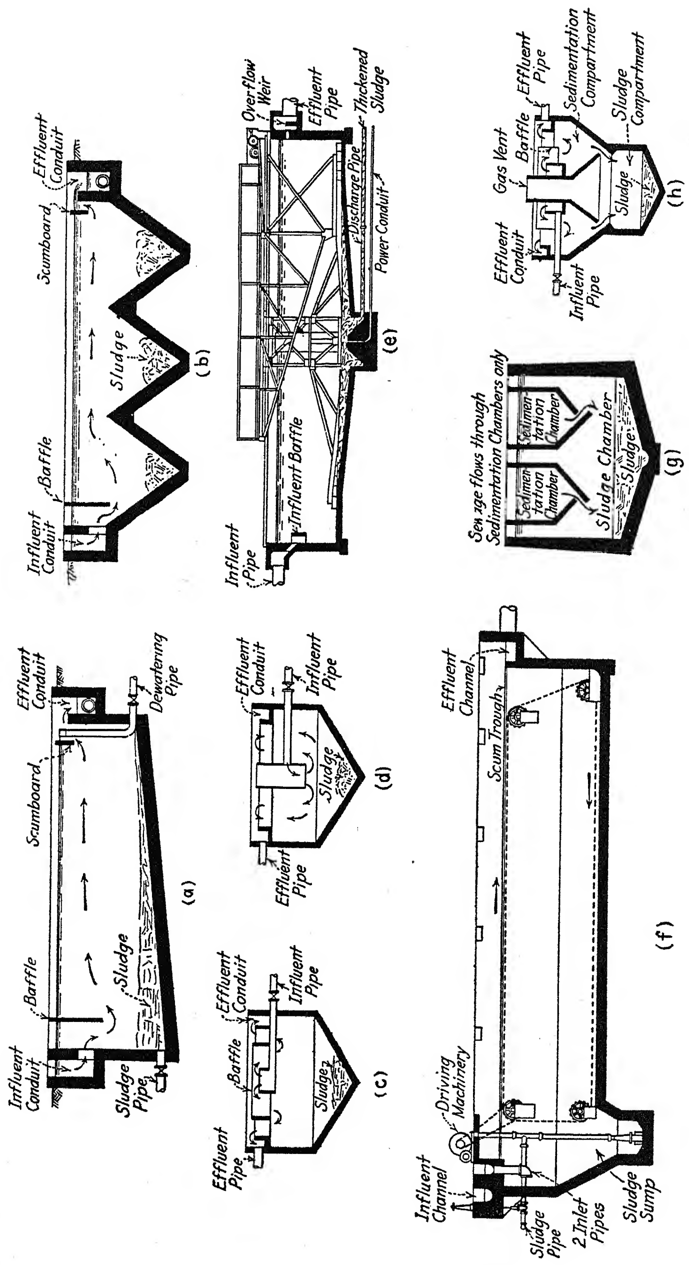
SEDIMENTATION BASINS AND SLUDGE DIGESTION TANKS

344. Classification of Sedimentation Units.—As stated in Chapter XIV, the object of sedimentation is the removal of the settling solids from the sewage by gravity and by aggregation of particles incidental to the subsidence of flocculent sewage matters. If coagulating chemicals are not added to the sewage, the structures in which the process of settling takes place are known as *plain sedimentation tanks.* If coagulating chemicals are used for the purpose of bringing the finer suspended and colloidal solids together into masses of larger bulk, thus increasing gravitational deposition of both coarse and fine solids, the structures are called *chemical precipitation tanks.* If the precipitated sludge is stored (1) in the bottom of the settling tank, (2) in a compartment situated below the flowing-through chamber, or (3) in a separate tank, the structures providing for this supplementary process of septicization, or sludge digestion, are respectively designated (1) single-story septic tanks, (2) two-story septic tanks, or (3) separate sludge-digestion tanks.

Without going into the matter of fill-and-draw tanks, which are now practically obsolete, the structures may be classified as follows:

(*A*) According to the direction of flow.
- 1. Horizontal-flow tanks.
  - (*a*) With continuous longitudinal displacement of the flowing sewage from the influent to the effluent end (Fig. 173 *a, b, e, f, g* and *i*).
  - (*b*) With continuous radial displacement of the flowing sewage from a central inlet to a peripheral outlet (Fig. 173 *c* and *h*).
- 2. Vertical-flow tanks with continuous vertical, and to a slight extent radial, displacement of the flowing sewage from a central downward directed inlet to a peripheral outlet, *Dortmund* tanks (Fig. 173 *d*).

(*B*) According to the method of sludge collection and removai.
- 1. Flat-bottomed tanks which must be emptied when the sludge is to be removed (Fig. 173 *a*).
  - (*a*) Without septic action.
  - (*b*) With septic action.



Influent Conduit
Baffle
Scumboard
Effluent Conduit
Sludge
Sludge Pipe
Dewatering Pipe
(a)
Influent Conduit
Baffle
Sludge
Scumboard
Effluent Conduit
(b)
Effluent Pipe
Baffle
Effluent Conduit
Influent Pipe
Sludge
(c)
Effluent Pipe
Effluent Conduit
Influent Pipe
Sludge
(d)
Influent Pipe
Influent Baffle
Discharge Pipe
Power Conduit
Overflow Weir
Effluent Pipe
Thickened Sludge
(e)
Influent Channel
Driving Machinery
Sludge Pipe
2 Inlet Pipes
Sludge Sump
Scum Trough
Effluent Channel
(f)
Sewage flows through Sedimentation Chambers only
Sedimentation Chamber
Sedimentation Chamber
Sludge Chamber
Sludge
(g)
Effluent Conduit
Influent Pipe
Gas Vent
Baffle
Effluent Pipe
Sedimentation Compartment
Sludge Compartment
Sludge
(h)

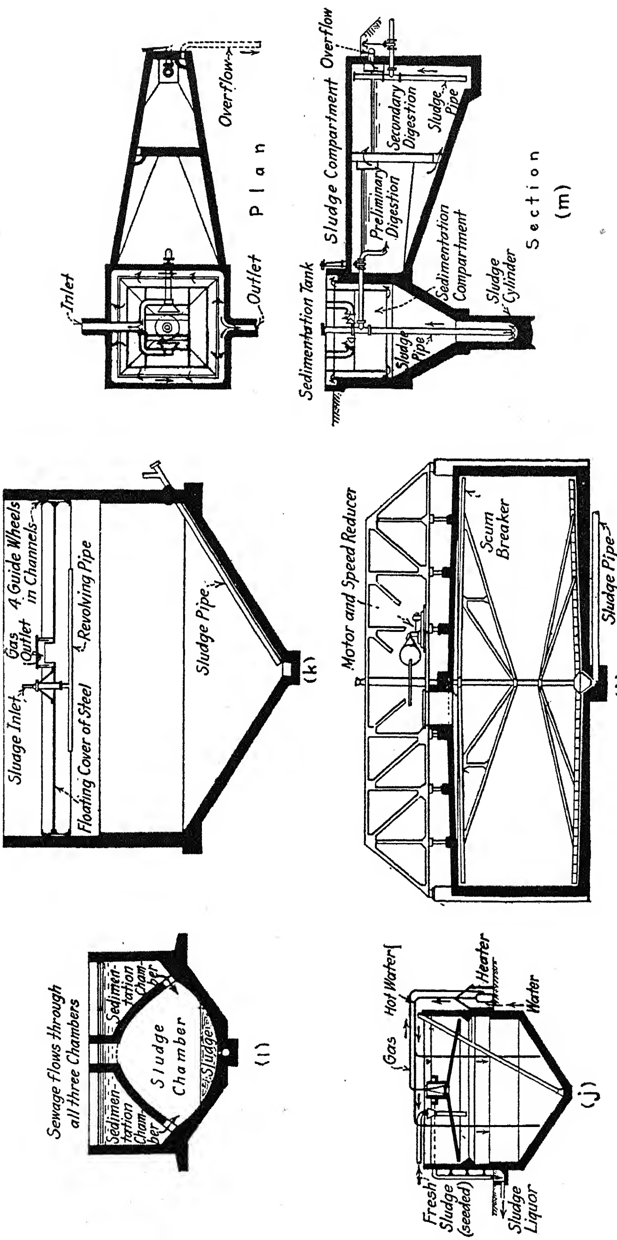


FIG. 173.—Various types of sedimentation tanks for sewage.

2. Hopper-bottomed tanks in which the sludge is withdrawn from the apex of the hopper, commonly during operation and under the influence of the hydrostatic head on the sludge (Fig. 173 *b*, *c* and *d*).
   (*a*) Without septic action.
   (*b*) With septic action.
3. Flat-bottomed tanks provided with mechanical cleaning devices in the form of scrapers or plows which move the sludge to a collecting point whence it is generally withdrawn as under *B*2 (Fig. 173 *e* and *f*).
4. Two-story tanks in which the sewage flows through upper sedimentation compartments, the settling solids sliding into lower sludge compartments, where they undergo septic decomposition.
   (*a*) With rigid separation of sewage and sludge, *Imhoff* or *Emscher* tanks (Fig. 173 *g* and *h*).
   (*b*) With partial separation of sewage and sludge, *Travis* or *hydrolytic* tanks (Fig. 173 *i*).
5. Combination of flat-bottomed, hopper-bottomed, mechanically-cleaned, or two-story tanks with separate sludge-storage tanks into which the deposited solids are delivered for septic decomposition, or digestion. (The digestion units are shown in Fig. 173 *j*, *k*, *l* and *m*.)
6. Shallow underdrained settling basins operated alternately as sedimentation units and as sludge-drying beds.

(*C*) According to the function and position of the units in the treatment works.
1. Preliminary, primary, or pre-sedimentation units, which precede oxidation processes or disposal without oxidation treatment.
2. Final or secondary sedimentation units, which follow oxidation treatment. Succeeding trickling filters, these are known as *humus* tanks.

Conventionalized sections of the various types of units are shown in Fig. 173. The principles of operation and design of sedimentation basins and sludge-digestion tanks will be treated in this chapter, with the exception of final or secondary sedimentation units, discussion of which will be deferred to the chapters dealing with the processes in connection with which this class of units is used.

PRINCIPLES OF OPERATION

345. Plain Sedimentation Tanks.—Two types of plain sedimentation tanks are in common use, (1) horizontal-flow tanks with longitudinal displacement of the sewage and (2) vertical-flow tanks. The former are employed particularly for preliminary clarification preceding oxidation processes or preceding disposal without further treatment other than disinfection in certain

cases. The latter find most frequent application as clarification units following trickling filters or activated-sludge treatment. Radial flow with plain sedimentation is not much employed. Formerly plain sedimentation tanks were at times arranged in series to provide a long route through which the sewage had to travel, a sufficient time for clarification being thus afforded. This arrangement is now seldom attempted, and tanks are commonly placed in parallel and in sufficient number to secure an adequate detention period.

Horizontal-flow Tanks.—In the order of their development, horizontal-flow tanks may be classified as flat-bottomed tanks (Fig. 173*a*), hopper-bottomed tanks (Fig. 173*b*), and tanks with sludge-removing mechanisms (Fig. 173*e* and *f*). The distinguishing feature of these tanks is the method of sludge collection and removal.

The older type of horizontal-flow tank is commonly a shallow rectangular structure. The bottom rises towards the outlet and slopes transversely towards one or more drains running longitudinally through the tank. The tank must be emptied for cleaning. A draw-off pipe or floating arm and valve are provided to remove the supernatant liquid (Fig. 173*a*). If this pipe or valve is situated at the outlet end, the greatest possible quantity of water will be withdrawn. Sludge sluices are provided at the deep end where the heavier and greater part of the settling solids subside if the velocities are not too high. Part of the sludge moves to the sludge sluices by gravity; the remainder must be pushed down the sloping bottom by means of wooden scrapers and squeegees. Gates in the side walls of the tank are sometimes used to introduce flushing water from adjoining tanks and thus to supplement the work of the scrapers. Hose connections on water-supply lines may also be employed for this purpose.

To eliminate hand-cleaning, hopper-bottomed tanks were developed. In these tanks the bottom is shaped into a series of hoppers into which the sludge settles. The slopes of the hoppers are sufficiently great to permit the settling solids to slide to the bottom where the sludge outlets are situated. The sludge is withdrawn from time to time by gravity or by pumping. Two different arrangements are commonly used (Fig. 178): (1) vertical or inclined riser pipes ending near the bottom of the hoppers, and (2) horizontal sludge pipes connected to the apices of the hoppers. Sludge may be withdrawn during operation.

The latest development in plain sedimentation tanks is the provision of mechanical sludge-removing equipment. Two types of mechanism are in widest use: the Dorr mechanism (Fig. 173*e*) and the Link-Belt mechanism (Fig. 173*f*). The former has been developed for use with circular or square tanks, the latter for rectangular tanks. Plows or scrapers are employed to move the deposited sludge towards the outlet, whence it is withdrawn by gravity or pumping. Sludge may be removed continuously or intermittently, without placing the tank out of operation. A somewhat thicker sludge is said to result than in other plain sedimentation tanks.

Sludge removal mechanisms are employed chiefly in connection with (1) preliminary sedimentation, the sludge being removed to separate digestion tanks, and (2) the clarification of the effluent from activated sludge aeration tanks. In both cases it is commonly desired to remove the sludge before septic decomposition sets in.

Vertical-flow Tanks.—In vertical-flow or Dortmund[1] tanks (Fig. 173*d*) the influent pipe extends to a considerable depth below the surface, where the sewage is distributed at a relatively low velocity throughout the horizontal cross-section of the tank. The pipe usually ends several feet above the elevation of the sludge deposits, to avoid any agitation of the sludge by the incoming sewage—unless it is desired to have the sewage as it enters come in contact with the sludge, so as to take advantage of any influence the latter may have in promoting precipitation by attraction and coagulation. This is apparently of some value with certain industrial wastes.

After leaving the inlet orifice, the sewage spreads out as it rises in the tank, and its velocity is gradually reduced to a rate at which the particles of suspended matter are just held in equilibrium, neither rising nor falling. As sewage passes this zone of equilibrium the coarser solids are mechanically filtered out by the suspended mass, which increases in density, and coagulation occurs by the aggregation of particles. When this mass becomes sufficiently dense, portions drop out of the stratum and settle to the sludge at the bottom.

Sludge is drawn from the conical bottom, as in hopper-bottomed tanks, without requiring the emptying of the tank, and may be delivered by gravity at a considerable elevation above the bottom

[1] A town in Germany where this type of tank originated.

of the tank, because of the hydrostatic pressure of the sewage above it.

Vertical-flow tanks are particularly adapted to the clarification of the effluent from trickling filters, because it is desirable to reduce as much as possible the time of contact of the liquid and sludge, which is best accomplished in this type of tank. If shallow tanks are used, the suspended matter in such effluents, accumulating on the sides and bottom, will reduce the quantity of dissolved oxygen in the liquid passing through them. Vertical-flow tanks are also much used for the settling of the effluent from activated-sludge units, because the deposited sludge is readily withdrawn and thus permits aerobic conditions to be maintained in sewage and sludge (see Section 396).

346. Chemical Precipitation Tanks.—Chemical precipitation tanks do not differ materially in design and operation from plain sedimentation tanks, except that they must be so proportioned as to (1) permit the chemical reactions to take place as well as allowing the coagulated solids to settle and (2) provide increased storage capacity for the sludge, which is greater in bulk due to increased removal of solids and the presence of the precipitating chemicals. As pointed out in Chapter XIV, the use of chemical precipitation is of waning significance in the treatment of municipal sewage, although still of importance in dealing with certain kinds of industrial wastes. Flat-bottomed, horizontal-flow tanks have commonly been employed in connection with chemical precipitation. They are preceded by mixing devices in which the chemicals added to the influent are dispersed through the sewage. Baffled channels with round-the-end or over-and-under baffles have been installed most widely for this purpose. For methods of storing, handling, adding and mixing the chemicals the student is referred to textbooks on water purification.

347. Single-story Septic Tanks.—These tanks serve a twofold purpose by providing (1) for the deposition of the settling solids by sedimentation and (2) for the partial or complete digestion of the sludge prior to its disposal. Horizontal-flow, flat-bottomed or hopper-bottomed tanks are employed almost exclusively. Relatively large sludge storage is provided so that the deposited sludge may remain in the tank for a sufficient length of time to undergo decomposition or digestion before being withdrawn. As a result of bacterial activity anaerobic conditions are established in the sludge. The organic matter is

decomposed and gasification, liquefaction and mineralization take place. By gasification some of the sludge is lifted to the surface and a floating scum layer is formed, in which other floating solids are enmeshed. Sludge particles which are freed from gases at the surface lose their buoyancy and settle back to the bottom. An upward and downward movement of sludge is thus maintained. To prevent the escape of sludge particles in the effluent much longer detention periods are therefore required than in plain sedimentation tanks, and baffles must be more carefully designed.

The scum forming on septic tanks (single-story, two-story and separate sludge-digestion tanks) consists chiefly of the coarse suspended matter which tends to float. Its quantity depends mainly on the character of this suspended matter. If the sewage is fresh and the suspended matter is not much disintegrated, large quantities of scum are probable. The suspended matter brought to the surface of the sewage sometimes forms such a compact mass that the entrained gases can be liberated but slowly. Meanwhile the formation of more gas in the remaining sludge carries more suspended matter to the surface, increasing the thickness of the scum perhaps to 2 ft. or more in extreme cases, and it often projects 2 to 6 in. or more above the surface of the sewage. Under such conditions, especially in open tanks, the surface of the scum is likely to become dry and leathery, thus forming a fairly tight roof, sometimes cracked by the gas pressure below it. Molds and fungi often develop in this mass, binding it together, and eventually weeds may cover its surface. Much trouble has been experienced under some conditions, particularly in the South, from the breeding of flies in this mat, necessitating screening and treatment with oil or other insecticides. Digestion of solids is greatly retarded in the scum.

Except for small, usually private installations, in which single-story septic tanks find wide application, this type of tank has been displaced largely by Imhoff tanks or by plain sedimentation tanks combined with separate sludge-digestion units. The reasons for this loss of favor are: (1) the appearance of solids in the effluent, which may at times increase the suspended solids content of the effluent above that of the influent; (2) the septic character of the effluent, which often increases its oxygen avidity, is likely to render it offensive, and sometimes unfits it for further biological treatment; (3) the nature of the sludge removed, which

may contain a large proportion of insufficiently digested solids, unless tanks are operated in rotation and each tank, after the sludge-storage capacity has been reached, is allowed to remain idle for relatively long periods of time before the tank is cleaned; (4) the difficulty of cleaning the tanks, especially in the breaking up of the scum layer, which is sometimes several feet thick and very tough; (5) the greater prevalence of objectionable odors; (6) the better economy of other treatment methods, especially as regards the required tank capacity.

Single-story septic tanks are cleaned in much the same way as plain sedimentation tanks. When the sludge is to be removed the tank usually must be unwatered. It is often necessary to break up the scum by means of hose streams, to induce it to settle to the bottom and flow to the sludge outlets.

348. Two-story Septic Tanks.—The idea of separating the settling solids from the digesting sludge, in order to avoid the objectionable features of septic tanks while taking advantage of the desirable ones, originated at the Lawrence Experiment Station of the Massachusetts Department of Public Health. Development of a two-storied structure which would operate on this basis was first accomplished by Travis[1] at Hampton, England (Fig. 173 *i*); next by Imhoff[2] in the Emscher Sanitary District, Germany (Fig. 173 *g* and *h*). The essential difference between these two types of structures lies in the more rigid separation of flowing sewage and digesting sludge accomplished in the Imhoff tank. In the Travis design from one-sixth to one-eighth of the sewage is sent through the lower, or sludge digestion, compartment, avowedly for the purpose of (1) washing out excessive quantities of undesirable enzymes or toxic products of bacterial activity and (2) reseeding the sludge with bacteria contained in the fresh sewage and required for the digestion of the solids. Imhoff could not agree that desirable effects were produced by the flowing sewage introduced into the digestion compartment, and considered that the solids washed out of the lower story degraded the quality of the effluent.

A further difference in principle and design of the two tanks lies in the use by Travis of vertical slats or splines, called colloiders, which are hung in the sedimentation compartment for the purpose of collecting on their surfaces finely divided suspended

[1] Hampton or hydrolytic tank.

[2] Imhoff or Emscher tank.

matter or colloidal substances. In the light of experience with submerged contact aerators (Section 424), however, the area of the contact surface provided by Travis seems to have been too small to be of practical value. In the later designs of Travis tanks, colloiders were omitted.

In the United States Travis tanks have not been installed. In Germany both Imhoff and modified Travis tanks continue to be employed. Radial-flow Imhoff tanks (Fig. 173*h*) were at one time frequently constructed. Expense and construction difficulties, however, have been largely responsible for their decreased use.

Imhoff Tanks.—In Imhoff tanks sewage flows through the sedimentation chambers only, and the sole function of these chambers is the removal of the settling solids. These solids drop to the inclined surfaces of the floor of the sedimentation chambers and slide through slots into the sludge chambers. The slot is trapped so that no gases from the sludge chamber can rise into the sedimentation chamber. In the sludge chamber the solids undergo septic decomposition as in a septic tank. The gases given off by this decomposition rise through the portions of the sludge chamber which are extended above the top of the tank, called gas vents or scum spaces; and scum collects on the surface of the sewage in these vents just as it does on the surface of septic tanks, and often to a greater depth. On account of the depth of the sludge chamber and the long period of time the sludge is allowed to digest, the sludge from tanks of this type which are operating normally is more dense than that from plain sedimentation tanks or septic tanks. Theoretically, therefore, the tank carries on simultaneously and independently, by means of its two-story construction, the functions of both plain sedimentation tanks and septic tanks.

To secure uniform distribution of settled solids throughout the length of the digestion compartment and thus to utilize the storage capacity in greatest measure, provision for reversing the direction of flow through the tanks is commonly made and transverse obstructions are avoided, as far as structurally possible. Inlets and outlets must be designed alike.

One of the most important advantages of this form of tank is its production, when operating properly, of sludge which is easily disposed of (see Section 429). Sludge is removed in much the same way as from hopper-bottomed tanks.

In order to prevent particles of sludge or scum from penetrating into the sedimentation or flowing-through compartments, the sludge and scum must be maintained at a distance of at least 18 in. below and above the slots, respectively. The clear zone is called a "neutral zone."

A peculiar feature of the operation of Imhoff tanks is the production, at times, of so much gas in the sludge chamber that the sewage in the gas vents seems to foam or boil. This is called "foaming." Occasionally the gases bring up with them so much sludge that the scum overflows the tops of the gas vents. Foaming seems to be associated with acid conditions in the digesting sludge, such as are observed when (1) the tank is overloaded, (2) rapid digestion of the sludge accumulated during the cold winter months sets in as the weather becomes warmer, (3) acid or acid-forming industrial wastes are present in the sewage. Excessive growths of certain protozoa are often associated with foaming.

Heavy scum formation and foaming can be combatted by (1) drawing off sludge; (2) breaking up the scum with paddles or other mechanical means; (3) directing a jet of water, or better, of sewage pumped out of the tank, preferably from the digestion compartment, into the scum spaces. Acid foaming can be reduced, too, by liming. Circulation of sludge liquor to which lime may be added, if necessary, is promising of good results.

The gases of decomposition which pass into the gas vents are either permitted to escape to the atmosphere or are captured by covering the vents and providing them with gas hoods which commonly collect the gas under water (Fig. 179). The gases, which consist chiefly of methane, may be utilized for heating or power purposes. Only the combustible gases, of course, are of value in this connection.

The sludge chamber of an Imhoff tank is intended to retain its contents in a stagnant condition, without any circulation of liquid through the slots opening into it from the sedimentation chambers above. It is therefore desirable to design and operate the tank so that no material fluctuations of sewage level will occur, for such fluctuations, particularly if sudden, will cause differences of hydrostatic pressure and, consequently, surges of septic sewage up through the slots. Furthermore, if currents down through the slots exist to such a degree that considerable fresh sewage

enters the sludge chamber, there is a possibility of the evolution of hydrogen sulphide.

The danger of diffusion increases with the number of sedimentation chambers overlying a single sludge chamber. Care must be taken to have the inlets and outlets designed to give the appropriate proportion of flow to each sedimentation chamber. If the operating conditions are such as to require careful attention from the superintendent, it may be advisable to connect the sedimentation chambers at each end of the tank so as to maintain the same surface elevation in all of them.

349. Separate Sludge-digestion Tanks.—Instead of placing the digestion unit below the sedimentation unit as is done in two-story tanks, the units may be constructed side by side or at some distance from one another. The expense and construction difficulties sometimes incident to the great depth of two-story tanks are thus reduced. Any one of the common forms of settling tanks may serve as the sedimentation unit, the deposited solids being conveyed thence continuously or at regular intervals to the digestion unit.

The principles of operation of separate sludge-digestion tanks are similar to those of two-story septic tanks. In order to secure satisfactory results, however, due attention must be paid to a number of factors that influence sludge digestion, particularly: (1) seeding, (2) reaction and (3) temperature. In two-story tanks, seeding and reaction control are somewhat inherent in the design, and the digestion compartment in a measure is prevented from becoming cold in winter by the sewage flowing through the sedimentation compartment. Failure to appreciate the factors controlling digestion is largely responsible for the difficulties experienced with the early separate digestion tanks. Modern plants are planned so that (1) the incoming solids are thoroughly mixed with digesting sludge and pocketing of fresh solids is thus avoided; (2) the settled solids are introduced while still fresh, *i.e.* before acid fermentation has set in; (3) the digestion tank is maintained as warm as practicable. Since the digested solids drawn from the tank contain less water than the incoming fresh solids, provision is made to remove the excess sludge liquor accumulating in the tank. This liquor may be discharged into the influent of the sedimentation basin or passed through a small sand filter before joining the effluent from the plant.

A number of types of sludge-digestion tanks have come into use. The older installations consist of rectangular earth basins, sometimes called digestion pits, or flat- or hopper-bottomed tanks, similar to plain sedimentation tanks. The newer designs are exemplified by the Emscher, Springfield, Dorr and Kremer-Kusch separate digestion tanks, shown in Fig. 173. The Emscher digester (Fig. 173*j*) is circular in plan with hopper bottom. Fresh sludge is introduced at the top; excess sludge liquor can be drawn off at different elevations and digested sludge is removed from the bottom. The tank illustrated has a fixed cover and is heated by its own gases (see Section 363). The Springfield tank (Fig. 173*k*) is circular in plan with hopper bottom and is equipped with a floating gas cover and a special means for distributing the fresh solids.

The Dorr digester (Fig. 173*l*) is equipped with a sludge-removing mechanism which also acts as a scum-breaker and distributes the incoming solids. Provision for heating the tank may be made. The Kremer-Kusch tank (Fig. 173*m*), used chiefly in Germany, is trapezoidal in section and plan and is divided into two compartments. Following destruction of the more readily decomposed materials in the first compartment, the more resistant materials are finally digested in the second chamber. The first compartment is shallow with large surface area; the second deep with small surface area. This arrangement is believed by the designers to favor digestion. It should be noted, however, that the fresh solids are not seeded with digested material in this type of tank. The Kremer-Kusch digestion tank is generally used in connection with the Kremer sedimentation tank, a vertical-flow tank with a deep sludge cylinder designed to facilitate the withdrawal of fresh solids.

One of the advantages of separate sludge-digestion tanks is their applicability where digestion is to be provided at existing plain sedimentation plants or where existing facilities for sludge digestion have been outgrown while sedimentation is still adequate. They are thus used in connection also with two-story septic tanks. Another advantage is the greater ease and economy of heating the digesting sludge. In single- or two-story septic tanks most of the heat that could be supplied artificially to the sludge, if heating were attempted, would be dissipated and lost in the sewage.

350. Shallow Underdrained Settling Basins.—These basins are only of passing interest to American engineers. They were developed as emergency measures in post-war Germany for the economical and temporary treatment of sewage. They are constructed like sludge-drying beds (see Section 432) and sewage is flowed through them to a depth of 4 to 16 in. Two basins are commonly operated in series, one sludging up while the other still effects clarification. As each basin fills with sludge, it is by-passed and the sludge is allowed to dry before being removed. Rotation is thus employed.

351. Efficiency of Sedimentation Units.—The many factors affecting the efficiency of sedimentation tanks have been discussed in Chapter XIV, from a theoretical standpoint. Foremost among them is the detention period, the practical significance of which can be gaged by a comparison of the results obtained from settling basins in cities with sewage of different characteristics. Fig. 174 shows the removal of suspended solids for a number of different sewages and different basins during a given detention period. In estimating the sedimentation in large settling basins the authors prefer to employ as a general guide studies of this type rather than theoretical considerations. In doing so, the remaining characteristics of the tanks are naturally taken into account. A modified and often more convenient way of presenting the information obtained in sedimentation studies is shown in Fig. 175.

A study of Figs. 174 and 175 shows, among other things, the following: (1) the greatest proportion of sedimentation takes place during the first hour, and very little added clarification is obtained beyond the second and third hours; (2) the stronger the sewage the greater the proportionate relative removal of suspended matter; to reduce the suspended matter in the effluent to the same value, however, longer detention periods are required for strong sewage than for weak sewage; (3) by plain sedimentation it is possible to remove from 40 to 75 per cent of the suspended matter from sewage of average strength (300 p.p.m. suspended matter) in from 1 to 4 hours.

Other observations show that, in general, the fresher the sewage, the more rapid is its clarification because the suspended solids are as yet uncomminuted. In well-designed tanks from 80 to over 95 per cent of the settling solids may be deposited. Bacterial removal often approximates that of suspended matter,

but during warm weather multiplication of bacteria in the liquid and non-settling portions of the sewage may offset the removal by deposition. This is especially true of single-story

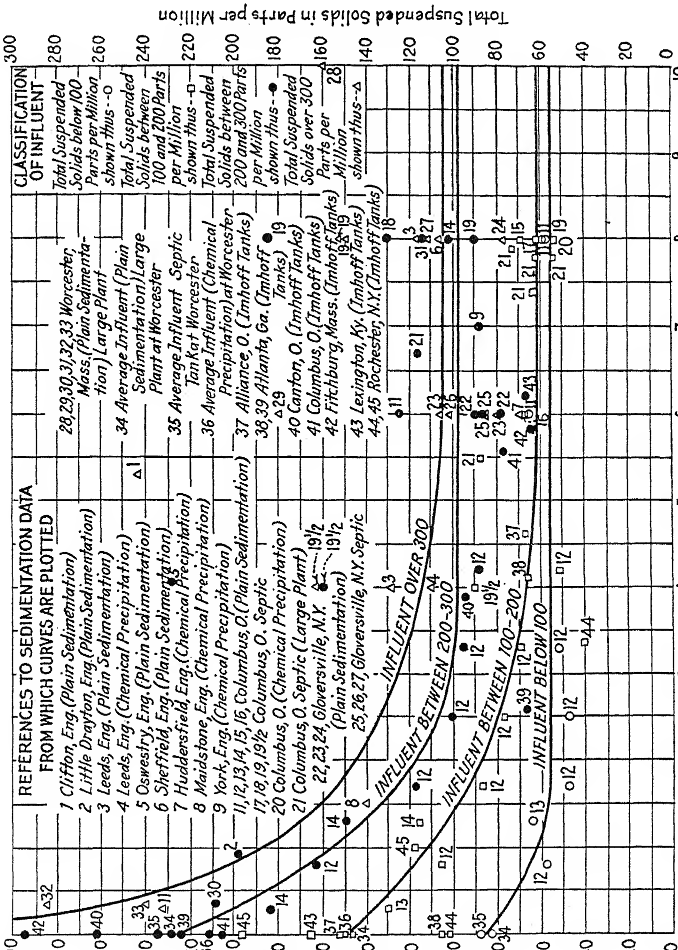


FIG. 174.—Removal of suspended solids from sewage by sedimentation.

septic tanks. Pathogenic organisms, of course, do not multiply. The removal of total organic matter is about half that of the suspended matter. Some studies of the reduction in the 5-day, 20°C., biochemical oxygen demand, by sedimentation in Imhoff

tanks, show values of 26 to 63[1] per cent with an average of about 40 per cent, but in general it is unsafe to assume so high a figure. Naturally, these results are not to be expected from single-story septic tanks.

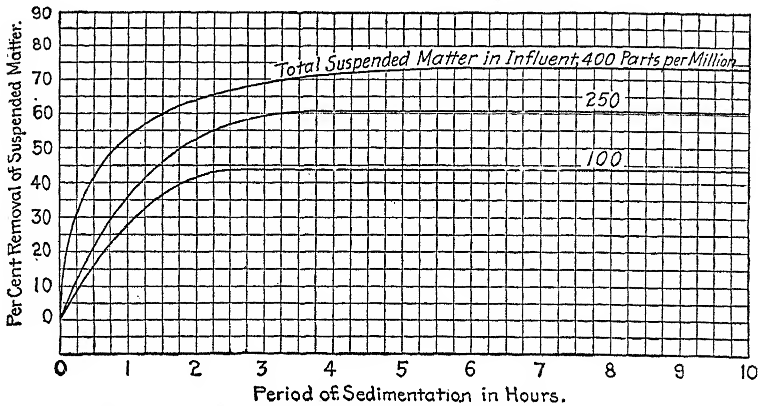


FIG. 175.—Percentage removal of suspended matter by sedimentation.

The progress of sedimentation in vertical tanks was studied experimentally by the authors at Gloversville, N. Y., where the

TABLE 83.—PERCENTAGE OF SUSPENDED MATTER REMOVED AT DIFFERENT DEPTHS IN VERTICAL TANK AT GLOVERSVILLE, 1909

(Averages of four experiments except as otherwise stated)

Upward velocity, ft. per hour	Depths of sedimentation				
	4 ft.	7 ft.	10 ft.	13 ft.	16 ft.
6	46	53	59	62	67
8	37	46	55	59	65
10	56	63	67	70	80
12	52	62	68	70	73
14	49	64	68	73	79
16*	37	51	63	64	65
16	..	..	55†	62†	66†
18	32‡	51‡	59	63	68
20‡	..	27	36	54	67
22	..	33†	57	64	74
24*	..	43	49	54	62

* One experiment only. † Average of two experiments. ‡ Average of three experiments.

[1] *Public Health Bull.* 132, p. 129.

sewage contains large quantities of tannery wastes which often cause a fairly good chemical precipitation and thus aid materially in the efficiency of the sedimentation process. Sewage entered the test tank 16 ft. below the elevation at which it flowed out, and samples of the sewage were drawn from the tank through faucets at different elevations. Table 83 gives the results of these tests, which indicate that the deeper the tank the more efficient the sedimentation at a given rate of flow. It will be seen that greater efficiency can be obtained by increasing the vertical height through which the sewage must flow than by increasing the area of the tank and decreasing the velocity and the height through which the sewage must rise.

352. Character and Quantity of Sludge Produced.—Besides the question of the removal of settling solids and other changes in the quality of the effluent of sedimentation tanks, the character of the sludge produced must be given consideration in a study of the relative efficiency of sedimentation processes. Among the factors to be weighed are (1) the volume of sludge to be stored and handled, (2) the ease of sludge drying and disposal, (3) the freedom from nuisances and (4) the possible recovery from the sludge of valuable constituents, either for the production of energy by utilizing gases of decomposition, or for fertilizing crops, or both. These questions must always be given consideration in the choice of sedimentation tanks and frequently control the selection. They are taken up in part in Sections 359 and 362 of this chapter and in part in Chapter XIX.

PRINCIPLES OF DESIGN

353. Detention Periods.—The detention periods generally employed in the design of sedimentation tanks are indicated in the following schedule:

1. Horizontal-flow, plain sedimentation tanks, including two-story septic tanks: ½ to 8 hours, average 2½ hours.
2. Vertical-flow, plain sedimentation tanks: ½ to 4 hours.
3. Horizontal-flow, chemical precipitation tanks: 2 to 8 hours.
4. Horizontal-flow, single-story septic tanks: 24 hours.

When the sedimentation period (a) has been fixed, the tank capacity (c) can be computed if the rate of sewage flow (e) is known, due allowance being made for the space occupied by sludge deposits or intended for this purpose: $c = ea$.

In hopper-bottomed tanks, the capacity of the hoppers is not included in the sedimentation capacity. In Imhoff tanks some designers do not consider that there is appreciable horizontal displacement in the V-shaped portion of the sedimentation compartments. In Travis tanks, a portion of the flow, commonly one-sixth to one-eighth, is diverted through the sludge compartments, and allowances must be made accordingly. The quantity of sludge to be expected in each type of tank will be considered in Section 359.

Detention periods commonly are based upon average dry-weather flows. In combined systems from 1.5 to 3 times the dry-weather flow is commonly handled; quantities of storm flow in excess either are by-passed or treated in separate storm-water tanks. Since the fluctuations of sewage flow are more pronounced in small plants than in large ones, detention periods commonly are increased by 50 to 100 per cent for populations under 5,000.

354. Limiting Velocities.—It is apparent from a consideration of the theory of sedimentation that the velocities ordinarily encountered in sedimentation tanks are of small moment as long as they are not sufficiently great to lift particles from the bottom or to move them along the bottom towards the outlet. Where sludge is in contact with the flowing sewage, lower limiting velocities are required than where the solids settle out of the sedimentation compartment into a lower sludge compartment. Velocities of 0.15 to 0.50 in. per second are common. Imhoff places the limiting velocity at 2 in. per second for Imhoff tanks. In vertical-flow tanks the upward directed velocity of the sewage must not be greater than the hydraulic subsiding value of the suspended solids that are to be removed. A limiting value of 0.03 in. per second generally is employed. In radial-flow tanks the upward velocity from beneath the annular baffle should usually not exceed 0.02 in. per second.

355. Tank Dimensions.—The tank dimensions to be considered are width, length and depth, which are frequently dictated by local subsoil conditions, especially by ground-water conditions. These affect the depth directly and the other tank dimensions indirectly. Construction of deep tanks may not be economical unless the ground-water table is low and ledge is not encountered near the surface.

Horizontal-flow Tanks.—For longitudinal displacement of the sewage it is customary to build tanks from 6 to 10 ft. in depth and

with a ratio of width to length ranging from 1:2 to 1:6. It is desirable even though more expensive to keep the width small in comparison with the length, in order to promote uniformity of distribution of the sewage on entrance and uniformity in its flow through the basin. The desired width can be secured by constructing a number of parallel basins, or by providing longitudinal baffle walls. In large basins uniformity of flow is affected by winds, by oscillations of the sewage due to gases escaping from the sludge, and by differences in the temperature of the sewage in different parts of the basin.

The length is governed by the detention period, the quantity of sewage to be handled, and the land area available for the basins. The tank area, as pointed out in Chapter XIV, is an important consideration. For granular solids, for which area is the controlling factor, Imhoff[1] has suggested a value of 1 sq. ft. for each 300 gal. daily. For sewage solids and depths greater than 5 ft. he considers that 1 sq. ft. for each 600 gal. daily will suffice, because sedimentation is due in part to the aggregation of the flocculent solids into masses of larger bulk, while they settle through the sewage.

The number of basins should be sufficient to afford the desired width and flexibility of operation by cutting in or out one or more basins. Even small works should be provided with at least two units, to permit cleaning and repairs without interrupting service. With the introduction of continuous sludge-removal devices square tanks have come into use. Radial flow in circular or square tanks is employed in but few cases.

Vertical-flow Tanks.—As previously pointed out, vertical-flow tanks are more or less restricted to employment following trickling filters and activated-sludge aeration tanks. Their use for preliminary clarification is so limited that standards of design have not been evolved. The tanks at Gloversville, N. Y., are about 49 ft. deep and 36 ft. in diameter; those at Toronto, Ontario, are 16 ft. deep, 25 ft. wide and 100 ft. long, with four hoppers in the bottom, each provided with a central, downward-directed inlet.

356. Inlets and Outlets.—The most obvious way to effect uniform distribution of the sewage entering and leaving horizontal-flow tanks is by means of inlet and outlet weirs extending entirely across the ends of the basin. But, while such we[illegible]

[1] "Taschenbuch der Stadtentwässerung," p. 46.

are satisfactory for the effluent, they are not satisfactory for introducing the sewage into the tank. This is chiefly because of the tendency of suspended matter to settle in the influent channel and to collect on the edge of the weir. Attempts have been made to solve this difficulty by cutting orifices or providing gated openings in the side of the influent channel, but these arrangements are generally unsatisfactory. The best method appears to be to provide a few relatively large openings through which the sewage will flow with moderately high velocity, which is checked almost immediately by small baffles placed in front

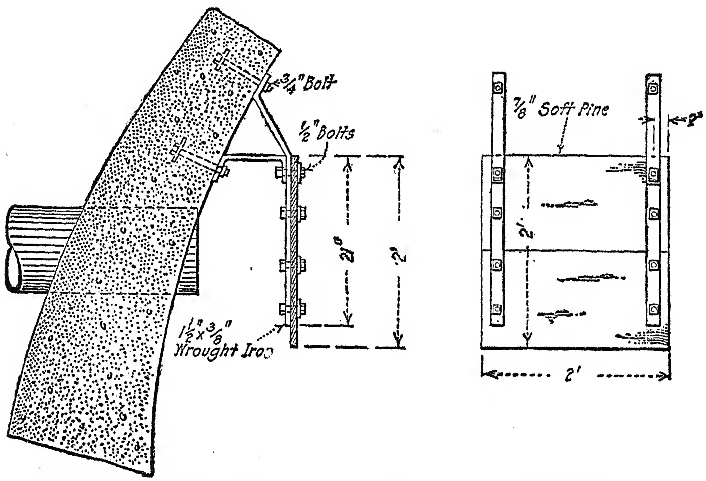


Fig. 176.—Baffle plate in front of inlet pipe of settling basin.

of and close to the influent openings. Fig. 176 illustrates such a baffle used by the authors in a covered sedimentation tank.

In the sedimentation compartments of Imhoff tanks, the depth and position of the inlet and other baffles should be so chosen that currents are not created in the lower V-shaped portion of the compartments. It should be kept in mind that in Imhoff tanks it is desirable to have the horizontal velocity gradually decrease towards the lower part of the sedimentation chamber, in order that there may be no currents to prevent the settling solids from passing through the slots into the sludge chamber.

Care is required in the design of inlet and outlet channels to ensure equal distribution of sewage and suspended matter among the different tanks. At Baltimore thin longitudinal walls were provided in the inlet channel and the individual channels thus

created were led to small groups of tanks. In the case of Imhoff tanks flow control is important, also, to limit the diffusion which takes place between the sludge and settling compartments when the surface of the sewage is not kept at a constant elevation (see Section 348).

The downward-directed inlet of vertical-flow tanks should be so dimensioned that the velocity of delivery is small. In order that the pipe might not occupy too much space, a small velocity was obtained at Gloversville by flaring the inlet at the bottom, the angle of flare being so chosen that particles settling on the outside would slide off into the sludge hopper.

In single-story tanks it is advisable to introduce and withdraw the sewage near mid-depth, in order to avoid disturbance of and interference by the accumulated sludge and scum.

357. Scumboards and Baffles.—To prevent the accumulation of floating matter on plain sedimentation tanks and its passage over the effluent weirs, in both plain sedimentation and septic tanks, scumboards are provided. These may take the form of (1) relatively shallow baffles which are fixed in position and rise above the sewage surface, or (2) floating boards which rise and fall with changes in the sewage level. Scumboards are commonly placed near the outlets and sometimes also near the inlets. Where the flow is reversible they must be provided at both inlets and outlets.

Where inlet weirs are used they are often guarded by scumboards from 2 to 4 in., or more in some cases, in front of them, extending to a depth of at least 2 ft. The narrow openings between the boards and weirs are easily kept free from scum, and the sewage is compelled by the depth of the boards to take the same course as it would follow with submerged openings.

Long plain sedimentation and single-story septic tanks are sometimes equipped with transverse baffles placed at equidistant points throughout the tanks, for the purpose of encouraging continued good distribution of the sewage as it flows through the tanks and to minimize the disturbances due to wind action, vortex motion, and other factors. Over-and-under baffles have been employed. Increasing the rate of flow by too much baffling will result in mixing and stirring-up of the sludge.

358. Bottom Slopes.—Bottom slopes should be chosen with a view to the satisfactory removal of the sludge. In flat-bottomed tanks, sludge free from grit can be moved with scrapers and

squeegees on slopes of 1.5 to 3 per cent without difficulty, but thick scum, which settles on top of the sludge when single-story septic tanks are drained, must be broken up before it can be handled easily on such slopes. In Great Britain a slope of about 6 per cent is considered desirable for the discharge of septic sludge by gravity. Bugbee states that with a grade of about 10 per cent the sludge from chemical precipitation tanks will readily flow to the sludge drains. It is often impracticable to provide such slopes in large tanks, however, for laborers are unable to move about readily on a sludge-covered slope steeper than 1 in 20. In such tanks a series of sumps with steep sides is necessary for the complete removal of sludge by gravity, and as such construction is expensive, it is customary to give the bottom a slope as great as the conditions of construction and operation permit and rely upon hand cleaning for the complete removal of the sludge. In order to facilitate the work, pipes with hose connections may be run between the tanks, so that the sludge may be flushed along with the aid of water, or sewage from one tank may be used in cleaning adjoining tanks. Tanks with mechanical sludge-removal are generally given bottom slopes of 8 to 15 per cent.

Experience at many plants has shown that where gravity is the only force relied upon to make the sludge slip down slopes, the latter should be at least 1.75 vertical on 1 horizontal, or better 2 on 1, in order to insure the feeding of sludge into the sludge pipes while sludge is being drawn. Sludge which contains gelatinous floc, often called colloidal matter by plant operators, has a tendency at times to adhere to very smooth surfaces as well as those of a rougher character, and wherever two surfaces form an angle the sludge tends to remain. It must be remembered that the slope of the valley at the intersection of two inclined surfaces is less than that of the surfaces themselves. Slopes sufficient to cause sludge to flow freely after the supernatant sewage has been drawn off are not necessarily sufficient to cause it to flow when the sludge is submerged, for the effective weight of submerged sludge in causing motion is less than that of exposed sludge by the weight of the displaced sewage. As a result, in drawing off sludge from tanks operated continuously, there is danger of forming a cone-shaped opening through the mass of sludge in the bottom of a tank and removing sewage as well as sludge, unless the bottom of the hopper is so formed by

steep sides that the sludge is inevitably concentrated in it. Trough-shaped bottoms rather than hopper bottoms have been used for the sludge chambers of some Imhoff tank installations.

To avoid excessive depths in large-sized hoppers, the slope is reduced commonly to 1 vertical on 1 horizontal, or even to 1 vertical on 2 horizontal. In a number of installations provision has been made for agitating and flushing the sludge with water. For Imhoff tanks the water may be admitted through a horizontal 1½- to 2-in. pipe at or near the top of the slopes, and occasionally through another ring or single jet in the sump at the bottom of the hopper. The pipe perforations are commonly about 5/32 in. in diameter and about 20 in. apart.

As it is essential for the settling solids to move steadily to and through the slots of two-story sedimentation tanks, the inclined sides of the bottom of the sedimentation chamber must have a steep slope and a hard, smooth surface. Slopes of 1.25 vertical on 1 horizontal commonly are used. Even with care in construction, it is necessary to clean the slopes from time to time with rubber squeegees, for soft, sticky material accumulates on them which will cause undesirable conditions in the chamber unless it is removed. For the same reason, it is desirable to have the top details of the tank such that this cleaning can be done readily, expeditiously, and thoroughly, either from permanently fixed platforms or temporary plank walks.

The slot at the bottom of the sedimentation chamber should be from 6 to 8 in. wide, depending upon the size of the chamber and the character of the sewage. The horizontal overlap to prevent gases passing through the slot should be at least 4 in. and preferably more.

359. Sludge Storage.—The volume of sludge for which provision must be made in sedimentation tanks or connected sludge digestion units depends upon: (1) the character of the sewage received by the tanks—its strength, freshness, preliminary treatment in grit chambers or by screening, and the addition of precipitating chemicals; (2) the degree of sedimentation effected in the tanks; (3) the condition of the deposited solids,—their specific gravity, water content, and changes in volume under the influence of digestion, tank depth or mechanical sludge-removal devices; (4) the period of sedimentation and sludge storage between sludge-drawing operations. Average values of sludge storage and the character and quantity of sludge drawn from

TABLE 84.—AVERAGE SLUDGE STORAGE TO BE PROVIDED IN DIFFERENT TYPES OF SEDIMENTATION UNITS AND CHARACTER AND QUANTITY OF SLUDGE DRAWN

Sedimentation process	Type of sewage	Sludge storage			Character and quantity of sludge drawn				Remarks
		Maximum time	Cu. ft. per million gallons sewage	Cu. ft. per capita	Per cent water	Specific gravity	Gal. per million gallons sewage	Lb. dry solids per million gallons sewage	
Plain	Separate	1–15 days	330	0.033–0.50	95	1.020	2,500	1,080	
Chemical precipitation	Separate	15 days	670	1.0	92.5	1.070	5,000	3,300	Depending upon type and quantity of chemicals used. Seventy pound of copperas and 600 lb. of lime assumed per million gallons.
Septic in single-story tanks	Combined	9 months	120	3.2	90	1.040	900	810	
Septic in two-story tanks	Separate	6 months	80	1.5	85	1.020	400	550	Depending upon temperature of digestion compartment and frequency of sludge drawing. Northern lat.tudes assumed. For southern latitudes capacity may be reduced by about 50 per cent.
	Combined		130	2.4	85	1.040	500	690	
Plain with separate sludge-digestion	Combined	6 months	130	2.4	85	1.040	500	690	Depending upon temperature of digestion compartment and frequency of sludge drawing.

different types of sedimentation units are shown in Table 84. They are presented with a view to serving as a general guide. Departures from the stated amounts are common and often appreciable.

In order to show the factors involved in determining upon the capacities to be provided, a discussion of some of the questions of sludge storage is given below, together with sample computations.

Plain Sedimentation Tanks.—In the hypothetical analysis of a typical sewage of medium strength, Fig. 128, it was assumed that the suspended solids amount to 300 p.p.m., of which 150 p.p.m. are capable of settling in tanks affording a 2-hour detention period. Still larger portions are capable of settling in longer periods. In general, the quantity of solids removed by passing such sewage through grit chambers and fine screens may be considered as equivalent to about 30 p.p.m. It is probable that as much as 10 p.p.m. of grit and coarse suspended matter in the original sewage would not be reported in an analysis, and would not be included in the 300 p.p.m. assumed in the hypothetical analysis. Therefore, of the 150 p.p.m. of settleable solids reported in an analysis 20 p.p.m. only will be removed by the grit chambers and screens, leaving 130 p.p.m. capable of settling in suitable tanks.

Plain sedimentation tanks yield so-called *fresh* sludge. On the basis of 90 per cent moisture the specific gravity of the deposited solids varies with the type of the sewerage system and the strength of the sewage about as follows:

Type of sewerage system	Strength of sewage	Specific gravity
Strictly domestic	Weak[1]	1.02
Strictly separate	Average	1.03
Combined	Average	1.05
Combined	Strong	1.07

[1] Due to infiltration or high water consumption.

Assuming 130 p.p.m. of suspended matter capable of settling in 2 hours, as an average, in accordance with the preceding statement, the weight of the dry solids deposited per million gallons is $130 \times 8.33 = 1{,}080$ lb. If the sludge contains 95 per cent water and its specific gravity with 90 per cent moisture is 1.03, we have

by Section 322 a volume of $\frac{1,080}{8.33 \times 0.10 \times 1.03} \times \frac{10}{5} = 2,500$ gal. of sludge per million gallons of sewage. On the basis of a daily per capita sewage flow of 100 gal. the sludge storage capacity must be 33 cu. ft. per 1,000 population for each day elapsing between sludge removals. These calculations give also the volume of sludge that must be digested dewatered or disposed of, as the case may be.

Chemical Precipitation Tanks.—The amount of sludge deposited in chemical precipitation tanks is greater than that obtained by plain sedimentation (1) because of increased sedimentation due to longer detention periods and the coagulating effect on the suspended matter of the chemicals added and (2) because of the presence in the sludge of the insoluble products of reaction of the chemicals used.

For purpose of illustration let us assume the following:

1. Solids removed in 8 hours, 210 p.p.m. $= 210 \times 8.33 = 1,750$ lb. per million gallons.

2. Copperas added, 70 lb. per million gallons forming $70 \times \frac{106.9}{278} = 27$ lb. ferric hydroxide (see Section 316).

3. Lime added, 600[1] lb. per million gallons, enough to satisfy the requirements of the copperas and combine with the free CO_2 and bicarbonates present to form calcium carbonate (see Section 316).

(*a*) From reaction with copperas, $70 \times \frac{112}{278} \times \frac{100}{56} = 50$ lb. $CaCO_3$ per million gallons.

(*b*) From reaction with CO_2 and bicarbonates, $\frac{300}{112} \times \left(600 - 50 \times \frac{56}{100}\right) =$ 1,530 lb. $CaCO_3$ per million gallons.

(*c*) Solubility of $CaCO_3$, 11 p.p.m. $= 11 \times 8.33 = 92$ lb. $CaCO_3$ per million gallons.

(*d*) Total $CaCO_3$ in sludge, $50 + 1,530 - 92 = 1,488$ lb. per million gallons.

4. Total solids in sludge on a dry basis $= 1,750 + 27 + 1,488 = 3,265$ lb. per million gallons.

Assuming a specific gravity of the sewage solids of 1.3 (because much of the light material is precipitated) and specific gravities of ferric hydroxide and calcium carbonate of 3.4 and 2.7 respectively, the specific gravity of the solids becomes 2.0, and since sludge of this type commonly contains 92.5 per cent water, the specific gravity of the sludge may be taken as 1.07, as indicated

[1] These are quantities used for many years at Providence, R. I.

in Table 84. The volume will be $\frac{3,265}{8.33 \times 0.075 \times 1.07} = 4,900$ gal. per million gallons of sewage.

Single-story Septic Tanks.—The organic matter in the sludge in septic tanks undergoes such changes that a reduction in its volume is inevitable, but this reduction is by no means so large as was claimed by early advocates of septic treatment. It is 10 to 40 per cent, the average being about 30 per cent. A considerable part of the reduction in volume is due to an increase in its density, following its disintegration and the compacting, resulting from prolonged stay in the tank. The sludge removed from a septic tank may be much less than 50 per cent of the volume removed from sedimentation tanks operated so as to prevent septic action. There is a great variation in the character of the sludge, due to variations in the quality of the sewage, temperature, construction of the tanks and methods of operation. Some sludge from septic tanks is reported to have very little odor but often it is decidedly offensive.

In computing the operating capacity of septic tanks, the space occupied by the sludge must be estimated by the methods followed in the case of a plain sedimentation basin and the result modified by the reduction of volume.

Provision should be made for storing about 60 to 70 per cent of the suspended matter not removed by coarse screens and grit chambers, plus its accompanying water. The total storage space to be provided will be fixed by the assumed lapse of time between tank cleanings, which depends in turn upon the condition of the sewage reaching the works, temperature and degree of digestion required.

There is always the probability that septic tanks built for small American towns will not receive proper attention, and usually such tanks are given large sludge storage capacity in consequence. With the fresh, weak sewage likely to be received at such plants and with small tanks there is more danger of large quantities of suspended matter in the effluent, due to high velocities of flow, than of producing a septic effluent, called over-septicization. Consequently it is well to provide storage for at least 9 months' accumulation of sludge consisting of 60 per cent of the total suspended matter and enough water to constitute about 90 per cent of the total weight, assuming that 30 per cent of the volatile solids (by weight) are digested. With plants for larger cities,

where fairly competent supervision is likely to be given, the provision for sludge may be reduced by designing the tanks so that a small amount of the oldest part of the sludge is drawn off at frequent intervals. This procedure is much favored in England, and the bottom slopes, sludge channels and sludge gates are designed to facilitate it. With frequent removal of the sludge, the percentage of water in it may be higher than when a long period of digestion is permitted.

With suitable allowances for differences in conditions, computations following those outlined under Imhoff tanks can be applied.

Imhoff Tanks.—The organic matter in the sludge of Imhoff tanks undergoes changes similar to those operative in single-story septic tanks. A theoretical method for arriving at the required storage capacity of the sludge digestion chambers is shown in Table 85. Here it is assumed, for purposes of illustration, that an hypothetical Imhoff tank situated in the northerly United States receives from combined sewage 100 lb. of deposited solids per month, of which 60 lb. are assumed to be organic and 40 lb. mineral matter. It is assumed also that 30 lb., or 50 per cent of the organic matter, must be digested in order to produce an inoffensive sludge. This is equivalent to 30 per cent of the total solids. For convenience this portion of the organic matter is designated *digestible solids*, although digestion is not confined to it. With separate sewage, the proportion of digestible solids would approximate 40 per cent of the total solids.

The temperatures of the digestion compartment are assumed to be those observed for the sewage at Schenectady, N. Y., in 1927. The rate of digestion is taken from Fig. 156, assuming that digestion is completed in 2 months at 60°F. and that the rate is constant. The latter is not strictly true, but errs on the safe side, as shown by Fig. 154. Since the solids freshly deposited in any month are not subjected to digestion for the whole month, one-half of the prevailing rate of digestion is assumed for these solids.

The sludge remaining in the tank at any time comprises the undigested solids contributed, plus the residue of digestible solids at the end of the month, deduction being made for sludge drawn during six months, May to October, inclusive, at a uniform rate per month, sufficient to remove all the digestible material at the end of October, except a small quantity remaining undigested

TABLE 85.—THEORETICAL COMPUTATION OF SLUDGE ACCUMULATION IN IMHOFF TANKS AT TEMPERATURES RANGING FROM 45° TO 64°F.

	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.
Temperature, °F	60	54	52	49	48	45	51	54	59	64	64	63
Digestion, per cent (Fig. 156)	50	36	32	27	25	17	30	36	50	63	63	59
Digestible Solids												
Solids deposited during month, lb	30	30	30	30	30	30	30	30	30	30	30	30
Rate of digestion, lb. per month												
Solids of stated month	7.5	5.4	4.8	4.0	3.7	2.5	4.5	5.4	7.5	9.4	9.4	8.8
Solids from previous months	15	10.8	9.6	8.1	7.5	5.1	9.0	10.8	15.0	18.9	18.9	17.7
Solids remaining at end of month, lb.												
From stated month	22.5	24.6	25.2	26.0	26.3	27.5	25.5	24.6	22.5	20.6	20.6	21.2
From 1 month before	6.2	11.7	15.0	17.1	18.5	21.2	18.5	14.7	9.6	3.6	1.7	2.9
From 2 months before	0	0	2.1	6.9	9.6	13.4	12.2	7.7	0	0	0	0
From 3 months before	0	0	0	0	0	4.5	4.4	1.4	0	0	0	0
From 4 months before	0	0	0	0	0	0	0	0	0	0	0	0
Total	28.7	36.3	42.3	50.0	54.4	66.6	60.6	48.4	32.1	24.2	22.3	24.1
Non-digestible Solids												
Solids deposited during month, lb	70	70	70	70	70	70	70	70	70	70	70	70
Solids drawn during month, lb	0	0	0	0	0	0	140	140	140	140	140	140
Solids remaining at end of month, lb	70	140	210	280	350	420	350	280	210	140	70	0
Total Solids												
Solids remaining at end of month, lb	99	176	252	330	404	487	411	328	242	164	92	24
Solids remaining at end of month, cu. ft. (average solids, 12½ per cent)	13	23	32	42	52	63	53	42	31	21	12	3
Solids remaining at end of month, cu. ft. per capita	0.39	0.69	0.96	1.26	1.56	1.89	1.59	1.26	0.93	0.63	0.36	0.09

and another portion required for seeding. The solids drawn amount to 140 lb. per month, or 840 lb. per year, 360 lb. having disappeared as the result of digestion.

The volume of sludge remaining in the tank has been computed on the assumption that the sludge will contain 15 per cent of solids at the bottom and 10 per cent at the surface, and will average 12.5 per cent.

The hypothetical data are reduced to terms in common use by assuming that the solids deposited in the tank correspond to 3 lb.[1] per capita per month (120 p.p.m. on the basis of 100 gal. per capita sewage flow). The tributary population, therefore, is 100/3. This computation indicates a required capacity for sludge storage of $63 \times \frac{3}{100} = 1.89$ cu. ft. per capita, the maximum being stored at the end of April. To this must be added an allowance for the neutral zone, *i.e.* the section of the tank between the surface of the sludge and the horizontal plane through the slots, which is not to be filled with sludge. The neutral zone is commonly assumed to occupy 18 in. of tank depth below the slots, say 20 per cent of the digestion-compartment volume below the slots, making a total of 2.36 cu. ft. per capita. The total sludge drawn per year (840 lb.), assuming 15 per cent solids, is equivalent to 2.7 cu. ft. per capita or 550 gal. per million gallons of sewage.

It is evident that an adequate knowledge of temperature conditions is essential to determining the sludge storage capacity of Imhoff tanks, as well as single-story septic tanks and separate sludge-digestion tanks. For rough computations the following approach is sometimes taken.

From analyses of the sewage, or by a study of the sources of the sewage when analytical records are lacking, the amount of suspended matter is first determined. For example, take this as 175 p.p.m., which is probable for domestic sewage collected by small separate sewerage systems. This quantity is equivalent to 1,460 lb. of suspended matter per 1,000,000 gallons of sewage. It will be assumed that a detention period of 2 hours is selected and that in this time 50 per cent of the suspended matter passes from the sedimentation chamber to the sludge chamber (see Fig. 173*g*). On this basis 730 lb. of solids will be collected in the sludge chamber from each million gallons of sewage. Some of this solid matter will be digested by the anaerobic processes

[1] For domestic sewage this would approximate 2.2 lb. per capita per month (88 p.p.m.).

going on in the sludge chamber, and this reduction of the solids will be assumed as 25[1] per cent, leaving 548 lb. of solids to go into the sludge. The water content of the sludge will be taken as 87.5 per cent. Sludge of this water content and with a specific gravity of its solids of 1.4, because of the digestion of the lighter organic solids, measures 0.452 cu. yd. per 100 pounds of suspended matter. Hence $5.48 \times 0.452 = 2.48$ cu. yd. of sludge storage capacity must be provided for every million gallons of sewage passing through the tank during the sludge digestion and storage period. This period may be taken in this case as 7 months, with the result that about 527 cu. yd. of space is required for a daily flow of 1,000,000 gal., or 1.42 cu. ft. per capita on a basis of sewage flow of 100 gal. per capita daily. No attempt should be made to compare this value with that of the preceding example, which was found for combined sewage; on the basis of separate sewage the preceding analysis would have yielded a value of 1.40 cu. ft. per capita.

In providing this amount of storage for the sludge, care must be taken that the shape of the tank will permit all the room allotted to the sludge to be actually available for it and that the sludge does not approach within 18 in. of the slots, measured vertically. Increasing the per capita value of 1.42 cu. ft. by 25 per cent we have 1.78 cu. ft. With some sewages the ebullition in the sludge chamber and the formation of scum are very active at times, particularly in protracted hot weather.

Another method of estimating the volume of sludge space has been developed by Kenneth Allen,[2] who has proposed the following formulas:

$$\text{Storage (combined sewage) in cubic feet} = 10.5\,PD$$
$$\text{Storage (separate sewage) in cubic feet} = 5.25\,PD$$

where P is the population in thousands and D is the sludge detention period in days. The formulas are based on fresh sludge with 90 per cent moisture and digested sludge with 80 per cent moisture, and the values must be correspondingly increased for lighter sludges.

Gas vents or scum spaces form an integral part of the sludge-digestion compartments of two-story tanks. The vents should be so designed that workmen can enter the sludge chamber

[1] Forty per cent of the sludge may actually be assumed to be digestible, but allowance must be made for the fact that the sludge deposited in the months preceding sludge-drawing is only partially digested.

[2] "Sewage Sludge," p. 228.

through them when the tank has been emptied. A neutral zone of about 18 in. should be maintained above the sludge slots, analogous to that below. The volume of the digestion compartment above the slots commonly becomes equivalent to ½ to 1½ cu. ft. per capita. When the digestion compartments are vented to the air the vents or chimneys are usually given an area equal to about 12 to 25 per cent of the tank area. Where the vents are small, the scum rapidly becomes thick, and if it is not broken up as soon as it collects in large quantities, it may rise over the edge of the vents. Imhoff recorded an instance of scum rising 6 ft. above the level of the sewage, because the gases in it could not escape. When the gas is collected only small chimneys are needed, leading to gas bells (see Section 362). It is to be noted that with certain industrial wastes, such as those from packing houses, the total amount of suspended solids may be so much greater than in the sewages ordinarily dealt with as to require abnormally large sludge compartments, scum spaces and gas vents and to necessitate radical changes in their design and departure from the usual proportions of Imhoff tanks.

Separate Sludge-digestion Tanks.—Computations for determining the capacity of separate sludge-digestion tanks are in all ways similar to those for Imhoff tanks. It must be kept in mind, however, that the temperature of the sludge in separate digestion tanks does not follow that of the sewage. If built above the ground, the temperature of the sludge more nearly approaches that of the atmosphere, and if built below the ground, that of the ground air or the ground water, depending upon the elevation of the water table. Since these controlling temperatures are frequently considerably lower in winter than those of the sewage, the space required is commonly larger than for Imhoff tanks. To reduce it, artificial heating must be resorted to (see Section 363). In deciding upon the tank capacity allowance must be made for the sludge liquor which enters the tanks when sludge is introduced.

360. Sludge Removal.—Methods of sludge removal vary with the different types of tanks and the means of sludge disposal. As drawn from settling tanks, sewage sludge will flow readily through pipes and open channels (see Chapter XIX) and can be pumped or caused to flow by gravity.

Flat-bottomed Tanks.—When a flat-bottomed tank is to be cleaned, the supernatant sewage may be drawn off by means

of a pipe extending from the bottom to the top of the basin and provided at the bottom with a swivel joint, Fig. 177*a*, which permits it to revolve through an angle of 90 deg. To the top of this pipe is attached a float of sufficient size to prevent the pipe from falling to the bottom, in which case it would draw sludge. The float is so adjusted as to hold the pipe a few inches below the surface of the sewage as it is drawn down, and a stop is provided so that when the pipe has fallen to a predetermined point it can go no farther. A gate should be placed in the pipe line with which the swivel pipe is connected so that the upright pipe can be kept partly filled, because its buoyancy will throw it up out of the water if empty. The upper end of the pipe should be enlarged

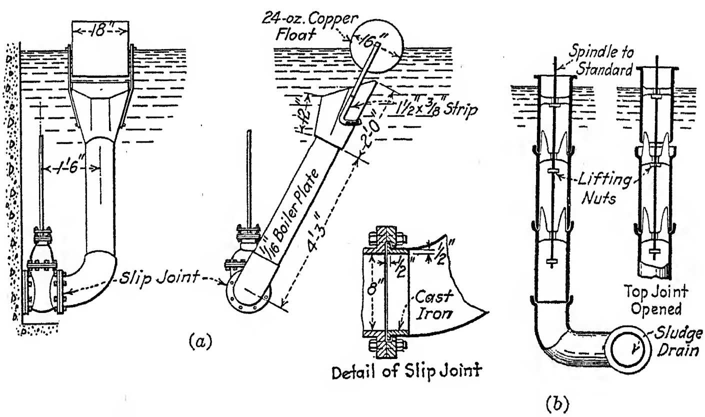


FIG. 177.—Equipment for unwatering flat-bottomed tanks.

and cut back at an oblique angle so that as it falls the opening will gradually approach a horizontal position and the sewage will be drawn from the surface.

In drawing off the contents of a septic tank by means of a swiveling arm of the type shown in Fig. 177*a*, the lip must be fitted with a scum board, and if the scum is tough or thick this form of draw-off pipe may not be as serviceable as that shown in Fig. 177*b*. This device has a vertical standpipe consisting of sections about 1 ft. long. Inside each section is a cross-arm with a hole in the center, through which passes a spindle raised and lowered by a gate standard of the rising-stem type. Lifting nuts are fixed on the spindle at such points that the top joint must be opened before the one next it, and the second joint must be opened

before the third. By using this detail there is a minimum disturbance of the scum and sludge. In case it is desired to operate with very thick scum, the first and second lifting nuts can be adjusted so that the top joint will not open.

The sludge is drawn off through sludge sluices situated at the deep end of the tank.

Hopper-bottomed Tanks.—Sludge is commonly withdrawn from hopper-bottomed tanks, whether single- or two-storied, during operation. Single-story septic tanks with hopper bottoms and operated on the principle of rotation, however, are usually emptied in order to remove sludge and scum. The sludge is drawn through pipes reaching from above through the tank to the bottom of the hoppers (Fig. 178*b*) or attached to them from below (Fig. 178*a*). On account of the hydrostatic head operative on the

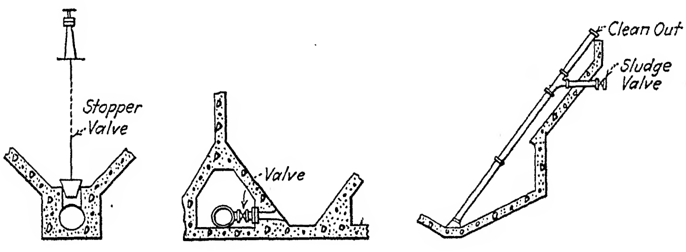


FIG. 178.—Details of piping for drawing sludge from hopper-bottomed tanks.

sludge inlets the sludge can be discharged at elevations lower than the flow line of the tank, allowances being made for the losses of head at entrance and the friction losses in the pipes. The minimum diameter of sludge pipes should be 8 in. to avoid clogging. If the sludge is discharged through a branch at the top of the sludge pipe the outlet of the branch should be from 4 to 6 ft. below the level of the sewage in the tank. If it flows through greater lengths of pipe a slope in the hydraulic grade line of 2 to 3 per cent should be provided.

In drawing off sludge, the rate of withdrawal should be slow in order that the whole mass of sludge may settle and no conical depression exist in the center. In the latter case a considerable quantity of relatively fresh sludge will be withdrawn from septic or sludge-digestion tanks and some of the older well-rotted material left, whereas all the sludge which is still undergoing active decomposition should remain in the chamber to keep up,

without interruption, the changes taking place there. Where gravity flow cannot be secured, the sludge must be pumped.

Mechanical Sludge-removal.—Sludge-removal mechanisms move the deposited solids to the sludge outlet during operation of the tank. They may be operated continuously or intermittently. The two types in common use are shown in Fig. 173*e* and *f*.

The Dorr mechanism consists of a group of revolving arms attached to a central motor-driven shaft, the motor either being stationary or running on a monorail on the walls of the tank. Plows attached to the arms move the sludge to the center of circular or square tanks.

The Link-Belt mechanism consists of a series of scrapers attached to endless belts running along the bottom of the tank, rising at the sludge-outlet end, passing over the tank and resubmerging at the opposite end. A square or rectangular tank may be employed, and the sludge is moved to a sump at one end of the tank, whence it is drawn.

361. Roofs.—In many small and medium-sized plants the sedimentation basins have been roofed, sometimes on account of severe winter weather and sometimes to conceal the tanks and their contents from public view. There is a small but indeterminate increase in the efficiency of the tanks caused by roofing them, but not enough to justify the expense of the covering. Open tanks should have their walls extend at least 6 in. above the surface of the liquid in order to prevent the sewage from being blown over the coping by winds. They should also be surrounded by fences if there is any danger of children falling into the tanks.

It was claimed by some of the early advocates of single-story septic tanks that they should be roofed in order to ensure proper anaerobic decomposition in the sewage and sludge, but this opinion is no longer general. The main reasons for roofing septic tanks or separate sludge-digestion tanks at present, particularly small tanks, are to keep them warm, collect the gases of decomposition, and reduce the odors often arising from them. If the wind is prevented by a roof from agitating the surface of the sewage, the odor is rarely so strong as to cause annoyance beyond the immediate vicinity of the basin.

Fixed roofs are sometimes made of wood but more often of concrete. For floating roofs wood, steel or concrete are generally used (see Section 362). When concrete is used the surfaces exposed to damp air should be finished as dense and smooth as

possible, in order to reduce danger of deterioration by the action of the hydrogen sulphide given off in the gases from the decomposing sewage and sludge. Vent pipes must be provided so that the sudden filling or emptying of a tank will not cause any large pressure above or below the roof. As the gases are sometimes explosive, a notice to this effect should be posted near the entrance to any roofed tank.

362. Gas Collection.—More and more sludge digestion units, whether of the two-story or separate type, are being equipped for the collection of the gases produced. These gases constitute one of the most readily utilized by-products of sewage treatment. They may be caught for the purpose of (1) heating the digestion

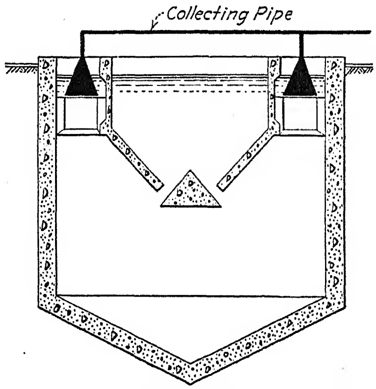


FIG. 179.—Gas-collecting equipment for Imhoff tanks.

units, (2) generating power, (3) being delivered into the municipal gas distribution system, and (4) being burned, to destroy odors given off by the digesting solids.

Two-story tanks lend themselves readily to the incorporation of gas-collecting devices. In modern installations gas-collection is frequently made part of the structural design either for immediate or anticipated use. Older plants, too, can easily be converted to include this feature. The partitions separating the sedimentation and digestion compartments form part of the gas-collection system and very little additional equipment is needed. The necessary arrangements are shown in Fig. 179. To prevent the clogging of the gas bells a scum barrier consisting of tongued and grooved boards set loosely, or of porous plates, is sometimes provided.

Separate sludge-digestion tanks may be equipped with stationary or with floating roofs. Stationary roofs are shown in Fig. 173*j* and *l*. A water seal prevents the escape of gas. Floating roofs (Fig. 173*k*) are designed on the same principle as gas holders. They are used both in connection with digestion tanks and digestion pits. The gas collectors at Birmingham, England, are constructed of reinforced concrete in the form of inverted pontoons that float on the digestion pits.

The amount and character of the gases depends very largely upon the character of the sewage. Between 500 and 700 liters of gas are commonly produced during complete digestion by a kilogram of volatile (organic) fresh sewage solids settling from domestic sewage (8.0 to 11.2 cu. ft. per pound). Between 70 and 80 per cent of this gas is combustible, being largely methane. The remainder is chiefly carbon dioxide and nitrogen. Under ordinary conditions of operation digestion is not carried to completion and only about 75 per cent of the total gas evolution takes place. The per capita gas production is, therefore, about 0.4 cu. ft. daily.[1] In the presence of large amounts of organic industrial wastes, the average value may, however, be exceeded two-fold or more.

A test[2] of gas production at Chicago is summarized in Table 86.

TABLE 86.—DATA ON GAS COLLECTION

(Imhoff Tank, Calumet Treatment Works, Average Results for 1 Year, 1926–1927)

Volatile solids removed, p.p.m.	37
Volatile solids removed, lb. per day	430
Gas produced, cu. ft. per day	1924
Gas produced, cu. ft. per pound volatile matter added	4.5
Gas produced, cu. ft. per capita per day	0.44
Composition of gas in per cent methane	76.6
Composition of gas in per cent carbon dioxide	14.7
Composition of gas in per cent nitrogen	8.2
Composition of gas in per cent oxygen	0.5
Heating value of gas in B.t.u. per cubic foot	780
Weight of 1 cu. ft. of gas, lb	0.054
Gas produced by weight from volatile matter added, per cent	24.3

[1] On the basis of a daily sewage flow of 100 gal. per capita and removal of 70 p.p.m. of organic settling solids, the daily per capita gas production (75 per cent complete) equals

$$\frac{0.75 \times 600 \times 70 \times 100 \times 3.78}{1{,}000{,}000} = 12 \text{ liters} = 0.42 \text{ cu. ft.}$$

[2] ZACK and EDWARDS, *Sewage Works Journal*, 1929; **1**, 173.

The average flow of sewage through the tank was 1.4 m.g.d. and the contributing population was 4,400. The storage capacity for sludge was 2.3 cu. ft. per capita. The Calumet sewage contains 116 p.p.m. of suspended matter, of which 55 per cent is volatile. The tank was operated without heat or pH control. The gas produced during the maximum month (September) and maximum day was respectively 2.52 and 3.3 times the average daily production. During the minimum month (March) and minimum day gas production fell respectively to 0.37 and 0.08 times the average daily quantity.

Data from a number of plants show variations in gas production of 0.21 to 0.71 cu. ft. per capita and 4.5 to 11.5 cu. ft. per pound of volatile solids.

363. Heating Sludge-digestion Tanks.—In order to reduce the sludge-storage capacity of digestion tanks which, as shown in Sections 321 and 359, is so largely dependent upon the prevailing temperatures of digestion, the tanks may be heated. The gases of decomposition may be used for this purpose. These gases may be burned under a boiler, either as collected or after scrubbing for the removal of carbon dioxide. The latter is readily absorbed in lime water or fresh water through which the gas is bubbled.

The calorific value of the gases of decomposition as produced varies from 6,000 to 9,000 calories per cubic meter of gas at 0°C. and 760 mm. pressure (700 to 1,000 B.t.u. per cubic foot).[1] This compares favorably with the heating value of commercial and natural gases in the United States.

The simplest method of heating digestion tanks is the addition of hot water to the tank or the circulation of hot water through coils placed in the tank. The dissipation of heat from the sludge compartments of two-story tanks is commonly so great as to render their heating impracticable. Heating is, therefore, generally confined to separate digestion tanks. The temperatures are seldom maintained above 70°F. because of economic considerations. When hot water is added it is usually brought to about 180°F.; when a circulating system is maintained temperatures of 140°F. are generally not exceeded, to prevent caking of sludge on the hot water piping.

[1] The calorific value of methane is commonly taken as 913 B.t.u. per cubic foot, measured at atmospheric pressure and 62°F. For values at other temperatures and pressures, see Marks: "Mechanical Engineers' Handbook," p. 375.

A separate sludge-digestion tank at Antigo, Wis.,[1] gave the following results during 1926 and 1927:

Average raw sewage temperature: day, 56.6°; night, 53.4°.
Average maximum air temperature: 56.5° (highest 96°).
Average minimum air temperature: 32.5° (lowest 23°).
Average sludge temperature: 70.3° (highest 84°).
Average heating water temperature, day: 96.5° in, 79.2° out.
Average heating water temperature, night: 90.6° in, 77.9° out.
Average heat exchange (24 hours): 14.3°.
Average heating water circulation: 1,700 cu. ft. per day.
Average heat added: 1,466,000 B.t.u. per day.
Average heat used: 444,000 B.t.u. per day.
Average heat lost: 1,022,000 B.t.u. per day.

Since the tank became full the average heat added has been 1,478,000 B.t.u. per day, and the heat lost 1,077,000 B.t.u. per day.

Average heat loss per pound of sewage per day (since tank was full): 0.6 B.t.u.

Average heat loss per square foot tank surface per day: 162.5 B.t.u.

Efficiency of gas boiler (taking 1 cu. ft. gas = 700 B.t.u.) was 65.7 per cent for 20-week period.

Efficiency of coal heater using gas as fuel = 31.8 per cent for 10 weeks.

364. Examples of Tank Design.—The features of design of a number of existing sewage-clarification tanks will serve to illustrate the principles set forth in the preceding sections of this chapter.

Plain Sedimentation Tanks.—This type of tank can be exemplified by (1) the flat-bottomed tanks at Marlborough, Mass.; (2) the hopper-bottomed tanks of the Passaic Valley Sewerage Commission; (3) the Dorr tanks at North Toronto, Canada; (4) the Link-Belt tanks at Lakewood, N. J.; (5) the vertical flow tanks at Gloversville, N. Y.

[1] Fischer and Quimby, *Public Works*, March and April, 1928.

TABLE 87.—COMPARISON OF PLAIN SEDIMENTATION TANKS

	Marlborough	Passaic Valley	North Toronto	Lakewood	Gloversville
Volume of sewage, m.g.d.	1.2	100	6	1	3
Number of tanks	2 2	12	2	3	2
Tank dimensions, ft.					
Length	30 62	225	50	60	36 (diam.)
Width	30 30	25	50	12	
Depth (maximum)	8 7	33	14	16	49
Number of hoppers per tank	....	9	1	1	1
Depth of hoppers, ft.	....	13	3.3	5.7	18.5, 29
Slope of floor	0 3 per cent	1.3 on 1	15 per cent	0	1.0–1.7 on 1
Detention period, hr.	5.8[1]	2.7	1.6	4	3.4
Frequency of sludge removal	6 times per year	3 times per week	Continuous	Continuous	Small amount drawn twice a week
Volume of sludge removed, cu. ft. per m.g.	205	182	....	....	1,085
Moisture of sludge, per cent	....	90	....	....	96

[1] Assuming all four tanks used for sedimentation.

Single-story Septic Tanks.—The gradual restriction of the use of single-story septic tanks to what may be termed home sewage disposal suggests a small installation as an example. A tank design used by the authors for residential sewage disposal plants serving 5 to 10 persons is shown in Fig. 148. The sedimentation compartment of this tank has a capacity of 500 gal., and the dosing tank of 115 gal.

Two-story Septic Tanks.—The following comparison of Imhoff tanks will illustrate the variations in design of some existing installations. Four installations are given: Schenectady, N. Y., Plainfield, N. J., Fitchburg, Mass., and Rochester, N. Y.

TABLE 88.—COMPARISON OF IMHOFF TANKS[1]

Item	Schenectady	Plainfield	Fitchburg	Rochester
Date of construction	1914	1916	1914	1916
Tributary population (1922)	65,000	40,000	38,000	260,000
Character of sewage.				
Flow, in million gal. per day	6	3.4	3.4	32
Separate or combined sewers	Separate	Separate	Combined	Combined
Strength (suspended solids), in p.p.m.	163	166	219	163
Freshness	Fresh uncomminuted	Stale	Fresh	Stale
Industrial wastes	Practically none	Practically none	Small amount	Small amount
Hardness of water supply, in p.p.m.	130	88	10	65
Temperature, in degrees, Fahrenheit	Average (1922) 56.1, maximum 64.7	46–70	No data	Average 54
Preliminary treatment				
Screening	Coarse rack	Fine screens, $\frac{1}{16}$-in. slots	Coarse racks	Fine screens. $\frac{1}{8}$-in. slots
Grit-chambers	None	None	Yes	Yes
Imhoff tanks				
Sedimentation period, in hours	3.3	3.4	6.4	1.1
Depth of tanks				
Total maximum water depth	13 ft. 9 in.	19 ft. 9 in.	24 ft. 8 in.	33 ft. 10 in.
Sludge compartment				
Depth below plane of slots	6 ft. 1 in.	8 ft. 9 in.	11 ft. 0 in.	22 ft. 6 in.
Depth below 18-in. neutral zone	4 ft. 7 in.	7 ft. 3 in.	9 ft. 6 in.	21 ft. 0 in.
Distance of lowest point of overflow below 18-in. neutral zone	2 ft. 6 in.	2 ft. 0 in.	2 ft. 0 in.	13 ft. 0 in.
Depth below overflow to adjacent compartment	2 ft. 1 in.	5 ft. 3 in.	7 ft. 6 in.	8 ft. 0 in.
Cu. ft. per capita	1.40	1.49	1.88	2.40
Number of hoppers	8	5	3	3
Ease of intercommunication	Poor (2-ft. square opening)	Poor (opening 20 by 24 in.)	Good	Good
Scum compartment				
Cu. ft. per capita	0.76	0.72	1.59	0.55
Area, gas vents; percentage of tank area	14.8	14.3	15	26.8
Area, gas vents; sq. ft. per cubic foot sludge capacity	0.034	0.023	0.031	0.0133
Loading, deposited solids				
Lb. per year per cubic foot sludge and scum space	14.2	10.4	13.1	9.2
Lb. per year per cubic foot sludge space	21.9	15.5	24.1	11.3
Lb. per year per cubic foot sludge space after deducting 18-in. neutral zone	30.0	18.9	30.9	12.3
Lb. per year per cubic foot scum space	40.4	32.1	28.5	49.7
Lb. per year per square foot gas-vent area	644	674	777	849

[1] *Trans.*, A. S. C. E., 1925; **88**, 465.

Separate Sludge-digestion Tanks.—The following installations are chosen for illustration: Antigo, Wis., Merchantville-Pensauken, N. J., and Boonton, N. J.

TABLE 89.—COMPARISON OF SEPARATE SLUDGE-DIGESTION TANKS

	Antigo	Merchantville-Pensauken	Boonton
Number of tanks	1	1	5
Tank dimensions, ft.			
Length	50 (diam.)	55	96
Width		40	12
Depth (maximum)	17.5	10	19
Slope of floor	2 per cent	0	1.4 on 1
Storage capacity, cu. ft. per capita	5	2	4
Method of heating	Water heated by collecting and burning gas	None	Provision for heating tanks by collecting and burning gas

Literature

EDDY, H. P. Imhoff Tanks—Reasons for Differences in Behavior. *Trans.*, A. S. C. E., 1925; **88**, 465.

GOULD, R. H. Comments on the Design of Sludge Digestion Tanks. *Proc.*, A. S. C. E., 1928; **54**, 2655.

Problems

1. The sewage of Fitchburg, Mass., contains on an average 200 p.p.m. of suspended matter, 100 p.p.m. being volatile solids. The average flow of sewage is 3.2 m.g.d. or 83 gal. per capita per day. Sedimentation units with a detention period of 2 hours are to be designed to meet present needs. Give your choice of number and dimensions of the units (including all floor slopes) and estimate the characteristics and amount of sludge to be handled, if the units are: (*a*) longitudinal-flow, hopper-bottomed, plain sedimentation tanks; (*b*) Dorrco square traction clarifiers; (*c*) Imhoff tanks; (*d*) combinations of tanks (*a*) or (*b*) with heated separate sludge-digestion tanks with floating gas covers.

2. Determine the per capita sludge storage to be provided in an Imhoff tank under the following average conditions: average sewage flow, 100 gal. per capita per day; total suspended solids in sewage, 200 p.p.m.; volatile suspended solids, 100 p.p.m.; detention period, 2 hours; temperature of sewage, as for Cleveland, Ohio (Table 68); winter storage of sludge, 6 months. Solve by theoretical method of Section 359 and check by rough estimate.

3. A separate sludge-digestion tank receives the solids settling in 2 hours from 1 m.g.d. of sewage containing 300 p.p.m. of suspended solids, two-thirds being organic. The tank temperature is to be maintained at 70°F. Estimate the required capacity of the tank (*a*) in cu. ft., assuming that sludge is to be stored during 6 months; and (*b*) in cu. ft. per capita, assuming a sewage flow of 100 gal. per capita daily.

4. (*a*) After a condition of equilibrium has been reached in a sludge digestion tank, what is the daily gas production of the tank in cu. ft. if its temperature is held at 70°F. and it receives a daily load of 2,500 gal. of fresh sludge with 95 per cent moisture, a specific gravity of 1.020 and a content of organic matter of 67 per cent (dry basis), assuming a gas production of 10 cu. ft. per pound of volatile solids? *Ans.* 7,130 cu. ft.

(*b*) If 80 per cent of the gas is methane, what is the heating value of the gas in B.t.u. per hour? *Ans.* 214,000 B.t.u. per hour.

(*c*) If during the coldest weather there is a daily heat loss from the tank of 5°F. and the incoming sludge has a temperature of 40°F., is the gas produced by digestion sufficient to maintain the tank temperature at 70°F., assuming that the tank holds 10,000 cu. ft. of sludge? *Ans.* Yes.

5. If the tank in Problem 4 is to be heated (*a*) by the addition of hot water, (*b*) by a hot-water circulating system, estimate (*a*) the amount of hot water that must daily be added to the tank, (*b*) the general characteristics of the recirculating system.

6. The gas production of an Imhoff tank at the Calumet treatment works of Chicago during 1926 and 1927 was as shown below. Make a study of the relation between temperature and gas production.

GAS PRODUCTION, CALUMET TREATMENT WORKS

Month, 1926–1927	Rate of gas production				Temperature, °F.		
	Average, cu. ft. per day	Ratio to yearly average	Maximum day, cu. ft.	Minimum day, cu. ft.	Air	Sewage	Sludge
September	4,840	2.52	6,400	3,300	67	58	70
October	1,495	0.78	2,300	1,000	54	58	66
November	1,607	0.84	2,000	900	39	48	62
December	1,390	0.73	1,700	500	28	48	61
January	1,375	0.72	1,700	600	26	44	56
February	804	0.42	1,200	150	38	45	51
March	706	0.37	860	500	44	44	47
April	1,205	0.63	1,700	800	51	48	48
May	1,221	0.64	1,800	500	58	53	52
June	1,815	0.94	3,400	1,000	68	58	58
July	3,884	2.02	5,100	3,300	77	69	62
August	2,749	1.43	4,600	440	70	69	67
Year, average	1,924	1.00	.....	.....	52	54	58

Population tributary to tank, 4,400.
Average gas per capita per day, 0.44 cu. ft.
4.50 cu. ft. of gas per pound of volatile matter added.

7. The data given under Problem 6 are to be applied in the design of a heated sludge-digestion tank. Make a study of the necessary heating arrangements.

CHAPTER XVII

SEWAGE FILTERS

365. Development of Sewage Filters.—The treatment of sewage by intermittent sand filtration is a natural outgrowth of the disposal of sewage by irrigation (see Chapter XIII). As such it was an attempt to intensify the activity of the forces of purification operative in natural soils by (1) more careful selection and preparation of the disposal areas and (2) better supervision of their operation. The thought of agricultural utilization of the filtration areas was not immediately relinquished. Cultivation or cropping, however, was made secondary to the securing of adequate treatment. Ultimately it was abandoned entirely. Development of the process of intermittent sand filtration is associated with the work of Frankland for the Rivers Pollution Commission of Great Britain (1870) and with the early investigations of the Massachusetts State Board of Health at Lawrence (1887).

Experiments by Dibdin in connection with the treatment of the sewage of London, England, are responsible for the next step forward in sewage filtration, the development of the contact bed (1892). Looking towards higher rates of treatment for the tremendous volume of sewage to be handled, and lacking suitable areas of natural sand deposits so essential to the economy of intermittent sand filtration, the behavior of filtering materials coarser than sand was studied. Trouble being encountered in operating the filters by dosing with sewage from above, a fill-and-draw or *contact* method of operation was devised and proved successful. These studies as well as some earlier investigations at Lawrence with gravel filters (1889) paved the way for the employment of still higher rates of treatment by causing sewage to trickle downward through a bed of coarse filtering material. Trickling filters[1] resembling present-day installations in construction and operation did not take form, however, until 1894. The names of Lowcock, Corbett and Stoddart in England are especially noteworthy in this connection.

[1] Also called "percolating," "aerating," "sprinkling" or "sprinkler" filters.

366. General Features of Sewage Filters.—The three types of sewage filters most commonly employed in America are (1) the intermittent sand filter where natural sand deposits exist, (2) the contact bed of coarse stone and (3) the trickling filter of coarse stone with sewage distribution by stationary nozzles.

Intermittent Sand Filters.—A typical group of intermittent sand filters is illustrated in Fig. 180. The filtration area, as here shown, is commonly broken up into a series of rectangular beds,

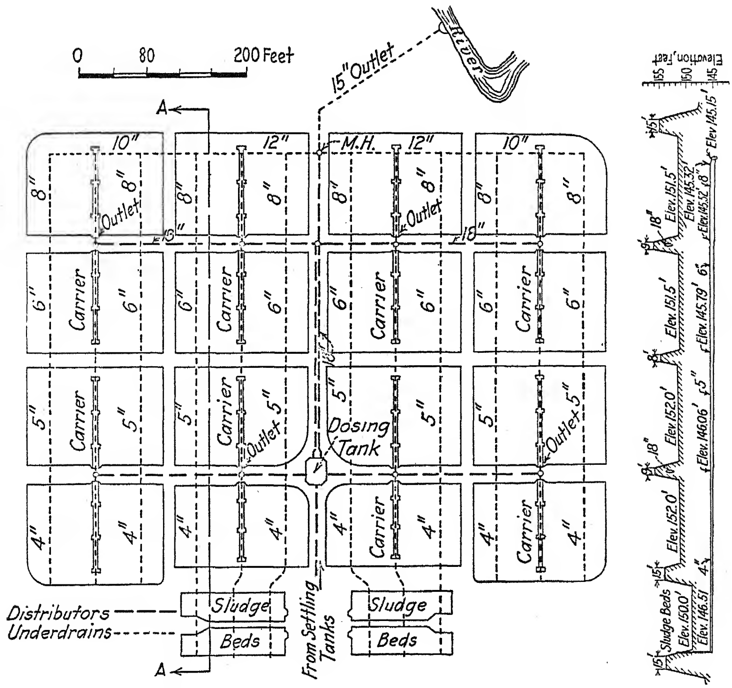


FIG. 180.—Arrangement of sand filter beds, North Attleboro, Mass.

successive rows of filters being separated by embankments, some of which carry roadways which give access to the beds for cleaning purposes. Sewage is discharged intermittently upon the sand surface, filters downward through the sand and is collected in underdrains. The receding sewage draws air into the sand and thus maintains aerobic conditions in the bed. Where deep deposits of sand and gravel occur underdrains may not be needed, in which case there will not be any plant effluent as such, the percolating sewage joining the ground water.

If there is much coarse material in the sewage this is strained out at the sand surface and forms a thin mat of sewage solids which intercepts finer suspended matter and becomes more and more compact. The filtering capacity of the bed is thus reduced, and it becomes necessary to remove the mat. This is commonly done by manual labor. In the absence of coarse solids and mat formation, when settled sewage is applied to the filter, for example, the finer particles penetrate farther into the sand and clog the top inch or more of sand which must then be scraped off from time to time. Where there are natural sand deposits, filter beds are constructed by removing the loam and top soil and bringing the surface to a level. In the absence of natural areas, the sand must be hauled into place. In some cases, crushed cinders are used instead of sand. The use of wholly artificial beds is, however, quite restricted.

Contact Beds.—Contact beds are made up of broken stone, cinders or similar inert material, packed into watertight tanks

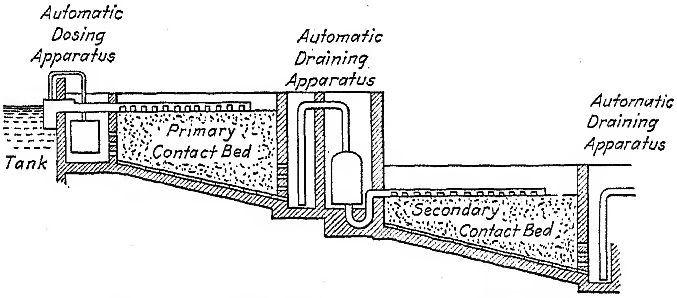


FIG. 181.—Arrangement of double contact beds.

or basins. The contact material is often called "ballast." Sewage is applied to the bed, commonly from above, the outlet from the bed being kept closed. The sewage fills the voids in the ballast and is permitted to remain in contact with it for a short time. The bed is then drained and allowed to rest, the voids now being filled with air drawn in by the receding sewage. The cycle is then repeated. As shown in Fig. 181 contact beds may be built in series, the effluent from the first or "primary" bed passing to a "secondary" bed. This is called "double contact." Triple contact has also been used. In double contact, the secondary beds usually can treat a somewhat greater quantity of sewage per acre than the primary beds, with satisfactory results. Triple

contact generally requires so much head for operation that it becomes uneconomical.

New contact beds have from 40 to 50 per cent voids. In the course of a few years, however, the voids generally become filled with sewage solids which are humus-like in character, and the contact material must then be removed, cleaned and replaced, or renewed. If relatively coarse ballast is employed the beds may "unload" much like trickling filters, and final settling is then required (Section 368).

Trickling Filters.—The filtering material employed in trickling filters is of the same general nature as that of contact beds. The

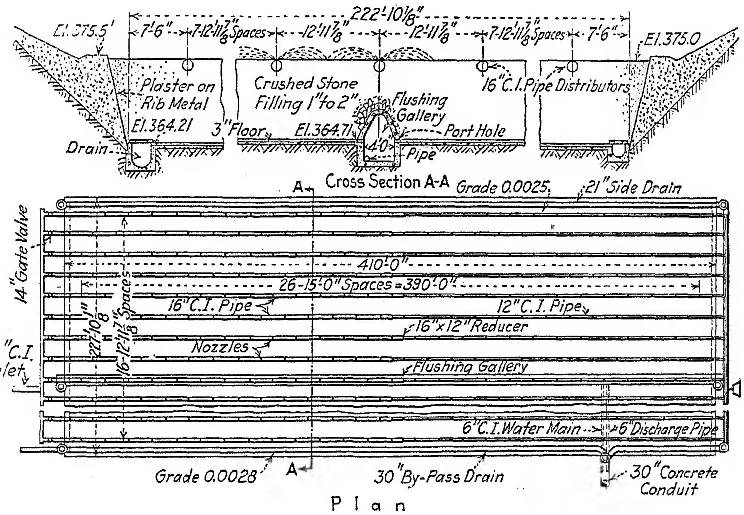


FIG. 182.—Arrangement of trickling filters, Fitchburg, Mass.

ballast is sometimes placed in water-tight tanks, sometimes not. Figure 182 shows a plan and section of the trickling filters at Fitchburg, Mass., built from the plans of D. A. Hartwell and the authors. Operating at a much higher rate than contact beds, trickling filters require a more elaborate system of sewage distribution and underdrainage. Sewage is applied to the surface by sprinkling devices of varying design, which will approach uniform dosing of the area. The sewage then trickles downward and is collected by underdrains. The pores of the bed are not filled with sewage, and ventilation is commonly such that oxygen

may be absorbed continuously by the jelly-like covering on the ballast and by the sewage, rather than intermittently as is the case with the older types of sewage filters. Dosage, however, is generally intermittent, but far more frequent than with intermittent sand filters and contact beds.

In further contrast to intermittent sand filters and most contact beds, which must be cleaned from time to time, well-designed and carefully-operated trickling filters are self-cleaning. They unload the accumulating, more or less stabilized solids, and hence the effluent requires secondary or final treatment in sedimentation tanks (Section 368).

367. Preliminary Treatment.—Some form of preliminary treatment is commonly employed in modern sewage filtration works. The purpose of this treatment is to reduce clogging in the case of intermittent sand filters and contact beds, and to prevent it in the case of trickling filters. The matter of nozzle clogging is also of importance in the latter type of unit. How far treatment should be carried will depend upon the economy of plant construction and operation. Grit chambers and coarse screens alone will ordinarily be insufficient for all but intermittent sand filters. Fine screens have been advocated in preference to sedimentation tanks in certain instances, although it is doubtful if the solids passing the screens but removed by sedimentation tanks can be economically handled by filters. If tanks are provided, they should be designed and operated to take out all the settling solids it is practicable to remove.

A disadvantage of the septic tank for preliminary treatment is that foul odors are often given off by the tank effluent. Spraying the effluent on trickling filters is particularly favorable to the dissemination of odors. There is also a general belief that sewage can be purified by bacterial agencies more advantageously when fresh than when septic.

Preliminary treatment is not so essential to proper operation of intermittent sand filters as it is to that of contact beds and trickling filters. Removal of the coarser suspended matter in fact results in deeper penetration of the remaining solids, necessitating the removal of greater depths of sand when the beds are cleaned. The advantages of using settled sewage, however, generally outweigh the disadvantages. In the operation of contact beds both raw and settled sewages have been applied. To avoid rapid clogging, however, pretreatment is essential.

Washing clogged filtering material should be avoided if possible. This is even more important in connection with trickling filters, which can be so designed and operated as to avoid clogging entirely.

Partial oxidation of clarified sewage prior to filtration will be discussed in Section 405.

368. Final Treatment.—The effluent from intermittent sand filters is clear and well nitrified and requires no further treatment. Fine-grained contact beds also produce a clear and stable effluent. The effluent from coarse-grained contact beds and trickling filters, on the other hand, usually contains considerable quantities of suspended matter.

Notwithstanding losses due to digestion, as much suspended matter may be expected in the effluent as in the influent in some cases; indeed, colloidal matter is often precipitated within the filter so that the effluent averages higher in suspended matter than the influent in such cases. This difference may be further augmented by bits of organic growths, worms, and the like washed from the filter.

In appearance, the suspended matter of trickling filter effluents is quite different from that in sewage. It is of a granular and gelatinous nature instead of the slimy, mucilaginous character of sewage sediment.

The increase in stability of the suspended matter in trickling filter effluent over that in the applied sewage was shown by an experiment at the Lawrence Experiment Station in which 0.2 g. of each sediment was mixed with 4,000 cc. of river water saturated with oxygen (1908 Report, Mass. State Board Health, p. 376). The results are given as follows:

	Dissolved Oxygen at End of 5 Days; Percentage of Saturation
River water	90.0
Trickling filter 135 sediment and river water	100.0
Trickling filter 136 sediment and river water	76.0
Sewage sediment and river water	1.5

It seems to be clearly established that the deposit from trickling filters is much more stable than that from sewage; yet it appears that the effluent may carry, at times of unloading, such large quantities of this matter that the water is rendered putrescible

thereby. Sludge resulting from the sedimentation of trickling filter effluents has usually been found to be putrescible. In some cases it has been rendered offensive by the presence of many decaying worms or by particles of organic growths.

In the great majority of trickling filter installations it will be necessary to remove suspended matter from the effluent by tanks, strainers or filters. Much of the suspended matter lends itself readily to sedimentation, but there is considerable fine matter which does not settle readily. For complete removal, strainers or filters must be used. In the latter case it will probably be economical to pass the trickling filter effluent through sedimentation tanks first. Intermittent sand filters will not only remove this suspended matter but will afford further bacterial purification and may be operated at comparatively high rates, as was done for a number of years at Gloversville, N. Y.

Where a highly treated effluent is required, sand filters have come into use also in connection with contact beds, as at Chatham, Morristown and Boonton, N. J. High rates of filtration are generally employed in such cases. The design and operation of final sedimentation tanks, often called humus tanks when used in connection with trickling filters, will be taken up in Section 388.

PRINCIPLES OF OPERATION

369. Filter Loading.—There is no really satisfactory way of expressing the permissible loading of sewage filters. The rate at which sewage may be treated to yield an acceptable effluent depends, among other things, upon (1) the quantity and nature of the organic matter in the applied sewage, (2) the character of the filtering material, and (3) the depth of the filter. It is not possible to combine into a single unit these factors which themselves are constituted of a number of variables. The most common methods of expressing filter loadings are, therefore, limited to a statement of (1) the volume of sewage that can be treated per unit of filter area or filter volume or (2) the population served per unit area or volume. Correct interpretation of filter loadings expressed in this way naturally requires familiarity with the factors listed above.

Under average conditions the rates of treatment shown in the following schedule are employed:

	Gal. per acre per day	Gal. per acre-foot per day[1]	Persons per acre	Persons per acre-foot
Intermittent sand filters				
Little or no preliminary treatment...	20,000– 75,000	....	400– 1,000	
Settled sewage.....	50,000– 125,000	....	500– 1,500	
Effluent from contact beds or trickling filters.......	100,000– 800,000	....	1,000–10,000	
Contact beds........	....	75,000–200,000	....	750–2,000
4-ft. bed, single contact............	400,000– 800,000	....	4,000– 8,000	
4-ft. bed, double contact.........	300,000– 500,000	....	3,000– 5,000	
Trickling filters......	....	100,000–500,000	....	1,250–5,000
6-ft. bed..........	1,000,000–3,000,000	....	7,500–30,000	
10-ft. bed.........	1,000,000–3,500,000	....	15,000–40,000	

[1] Gal. per cubic yard or per cubic foot is sometimes used instead of gal. per acre-foot. 1 gal. per cubic yard = 1,613 gal. per acre-foot. 1 gal. per cubic foot = 43,560 gal. per acre-foot.

It should be noted that the loading of intermittent sand filters is expressed, in this schedule, in terms of surface area only, whereas depth is taken into account in the case of contact beds and trickling filters. The reason for this will be explained in the next section.

Attempts have been made, with limited success, to express permissible filter loadings in terms of the organic constituents of the sewage. The nitrogen load, or quantity of nitrogen in the form of organic nitrogen and free ammonia which is applied per unit area or volume, is such a measure. If, for example, an average sewage contains 15 grams per capita daily (Section 249), the permissible nitrogen load on an intermittent sand filter treating raw sewage is about $700 \times 15 \times 0.0022 \times 365 = 8{,}400$ lb. per acre per year. In the absence of determinations of organic nitrogen twice the albuminoid nitrogen is sometimes used to approximate the former value.

370. Effect of Depth upon Loading.—There is shown in Fig. 183 the ammonia removal at different depths recorded by various observers for trickling filters receiving the same quantity of sewage per square foot of surface area. It appears from this plotting that the percentage ammonia reduction per foot of depth is practically constant; the quantity of ammonia removed per foot of depth naturally decreases, however, as the quantity remaining diminishes. The lower depths of the filter are therefore required

to do less work than the upper ones. Studies of nitrification effected yield similar information. Strictly speaking, therefore, the loading of trickling filters should not be based on the volume of ballast. This method of statement is subject to the further error that, with equal loading per cubic yard, deep beds receive more sewage per unit of surface area and are, therefore, more subject to surface clogging. Finally, filtration is an oxidation process, and the effectiveness of the lower filter strata depends to some degree upon proper ventilation.

Offsetting to some extent these disadvantages of greater depth is the fact that the time of contact seems to increase geometrically with depth. Thus analysis of Clark's[1] observations

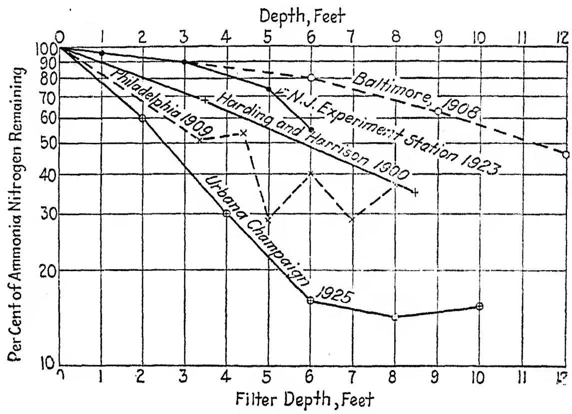


FIG. 183.—Effect of filter on ammonia reduction.

at Lawrence shows that for a given dosage of sewage per acre the time of contact is practically doubled for every increase in depth of 2 ft., and that a filter 10 ft. deep will have the same period of contact (about 1 hour) when receiving 2.5 million gal. per acre as a 4-ft. bed dosed at one-fifth this rate.

Clark's researches led him to conclude that the quantity of sewage which can be treated per unit area of bed increases at a greater rate than the increase in depth. This conclusion is based upon parallel tests in which a relatively uniform degree of purification was secured from filters of different depths. Unfortunately most, if not all, of the other tests which have been used as a basis of judgment upon this subject were not operated

[1] "Experiments upon the Purification of Water and Sewage," 1915, p. 22.

to secure such a uniform degree of purification, and the value of the tests to indicate volumetric efficiency is subject to doubt.

What has been said about trickling filters applies in a general way also to intermittent sand filters, excepting that in the latter the fineness of the filtering medium and the formation of a surface film of sludge concentrate the purifying processes in large measure at the surface. This concentration is sufficiently great to warrant basing the loading of sand filters, as indicated in the preceding section, upon the surface area of the bed.

The method of operating contact beds, however, is such that they can be loaded in direct proportion to their depth. Their activity does not change with depth as long as they are in good working condition. This stands in contrast to the other filtration methods.

371. Maturing of Beds.—All types of sewage filters are less effective in producing a good effluent when they are first placed in operation than after they have matured. The reason for this is that purification depends upon the changes which take place at the jelly-sewage interface and upon the activities of bacteria and other organisms living in the zoöglœal films which cover the filtering medium (Section 324). Time must elapse before the proper conditions are established and the bed has become "broken in," "mature," or "ripe." The period required varies with the season of the year from a few weeks in warm weather to several months during the cooler seasons.

During warm dry weather filter loadings might be appreciably increased, but generally the volume of sewage at such times is below the average for the year. The increased efficiency of the filter, therefore, is more likely to be utilized by the production of a better effluent, which is a favorable condition because at higher temperatures the oxygen demand of streams increases and their flow at such times is usually relatively low.

372. Clogging of Beds.—Sewage filters are more or less subject to clogging. This reduces their capacity and impairs the quality of the effluent. Clogging is expected in the operation of intermittent sand filters. Being restricted to the top layers of the sand, cleaning operations are relatively simple. Contact beds, on the other hand, may not clog for years, especially if the ballast is coarse and a certain amount of unloading takes place. Clogging, however, may occur at any depth and may require complete removal of the ballast for cleaning. Under normal conditions of

installation and operation, trickling filters should remain self-cleaning, and clogging should be an exceptional occurrence.

When sand beds are put in operation the areas adjacent to the distributor outlets clog first because (1) more sewage passes through the beds at these points and (2) the coarsest suspended matter is deposited nearest the outlets. At the beginning these areas, therefore, do more work than those near the periphery of the beds. As time goes on the opposite is true. Road dust carried by storm water is particularly objectionable from the standpoint of clogging. Rain compacts the sand surface, but moderate frost tends to open the pores of the bed.

The Royal Commission on Sewage Disposal recognized eight causes for the clogging of contact beds:

1. Disintegration of the ballast when beds are constructed of friable material.
2. Consolidation of the ballast, particularly when finer materials are used.
3. Deposition of colloidal material.
4. Excessive growth of organisms when beds are worked at high speed.
5. Excessive loading.
6. Insufficient rest.
7. Inefficient drainage.
8. Accumulation of solids, clogging varying directly as the quantity of suspended matter in the sewage.

Clogging of contact beds may proceed until the voids are completely filled, or it may stop after the bed has matured. The depth at which clogging takes place may shift up or down.

The periodic storage and unloading of solids by trickling filters is one of their most important attributes. Since the beds harbor huge numbers of living organisms, it is not surprising that they respond quickly to changes in temperature and other conditions. In summer, in the northeastern states, the quantity of solids in the effluent is about the same as in the applied sewage. Oxidation is then more active and nitrification will take place with doses not too great to permit efficient oxidation. A trickling filter designed and operated for that purpose will convert a large portion of the organic nitrogen into nitrates. In the fall, the efficiency of oxidation drops progressively with the lowering of the temperature, and the proportion of solids in the effluent to those in the sewage also is reduced. During the winter oxidation is low and much organic matter applied to the filter is stored in it. But with the first warm weather of spring, the filters emit great quantities of solids, far in excess of those in

the sewage applied at the time. Thus the stored matter, including the bacterial jelly which has served its purpose, is ejected from the filters, which thus recover capacity. Likewise, the renewed activity of the organisms produces more complete oxidation, and the quality of the effluent gradually improves until it equals that of previous summers.

Great numbers of worms develop in trickling filters, and are discharged with the accumulated organic matter when the filters "unload" these solids into the effluent. The abundance of these worms indicates that they must perform some function in the general transformation of organic matter into more stable matter which goes on inside the filters.

Normally the clogging of trickling filters is restricted to the surface, and "pooling" of the sewage may then occur. Surface clogging may be due to (1) too fine ballast or disintegration of the surface layer, (2) inadequate preliminary clarification of the sewage, or (3) growths of filamentous algae and fungi and the formation of a tough and impermeable mat.

373. Cleaning of Beds.—The cleaning of sand beds constitutes a part of the routine of their operation; contact beds and trickling filters, if properly operated, require much less frequent attention for this purpose.

Where raw sewage is applied to sand beds the heavy mats of sewage solids crack and curl upon drying, and may be peeled off in this condition or scraped off while moist. The mats contain much organic matter and have some small value as fertilizer. When the solids penetrate more deeply into the sand, as happens when settled sewage is applied to the beds, the clogged surface sand must be removed from time to time. This material seems to have little practical use and generally is disposed of by filling low-lying lands near the plant. Washing dirty sand for replacing is open to the objection that the finer sand particles are likely to be lost together with the foreign material; therefore the replaced sand is coarser than the original sand and permits greater penetration of fine solids, and their accumulation as a sealing layer at the surface of the undisturbed sand.

Clogged beds are sometimes harrowed or plowed. They should be carefully cleaned before this is done so that the solids which have been deposited upon the surface by the sewage will not become mixed with the sand and thus decrease the capacity of the bed. Addition of new sand to make up for the sand removed

during cleaning does not seem to have been employed much at municipal plants; some industrial installations, however, have resorted to this practice. Overtaxed beds in which fouling sets in because of insufficient aeration must be rested. This may take several weeks.

Contact beds which have become clogged after several years of operation (4 to 5 years is a common length of service) are cleaned by removing the ballast, washing it by hand or in machines often similar to concrete mixers, regrading it and replacing the clean material in the beds. Temporarily overloaded beds will recuperate during a resting period of shorter or greater length depending upon the circumstances.

There are several methods for combatting surface clogging of trickling filters and the resulting ponding of sewage:

1. Resting and drying the bed without permitting destruction of the contact film to take place.
2. Picking over, loosening, forking or harrowing the surface layer.
3. Chlorination of the applied sewage for the destruction of surface growths, or use of copper sulphate, caustic soda or chlorine compounds for this purpose.
4. Cultivation of the water springtail (*Achorutes viaticus*, a member of the family *Poduridae*), a wingless insect which feeds on the organic slime accumulating on the surface.

Clogged underdrains sometimes may be freed by flushing the drainage system by means of hose streams. Chlorination of the applied sewage may be effective in causing an unloading of the humus-like materials collecting on the ballast of the filter.

374. Odors from Sewage Filters.—Generally speaking, there is less opportunity for the dissemination of odors from contact beds than from intermittent sand filters, and less from the latter than from trickling filters. The intensity of the odors produced varies with the concentration, composition and condition of the sewage and with the weather (temperature, humidity and wind movement). In general, if the sewage reaches the plant in a relatively fresh condition, and if the filters are well operated, the odors should not be intense or especially offensive. Sewage odors are particularly noticeable on relatively calm damp days.

It is generally desirable to keep sand beds and trickling filters from ¼ to ½ mile distant from any substantial settlement. Small installations are sometimes covered to reduce the travelling of odors. Screening of the plant by stands of evergreen trees has been employed with questionable success, while chlorination

of the sewage prior to filtration has met with marked success, but destruction of the filter life by chlorine, must be guarded against. The hygienic significance of sewage odors does not appear to be measurable.

375. Filter Flies.—A small, gray moth-fly (*Psychoda alternata*) is sometimes very troublesome about trickling filters, but does not occur on sand filters or contact beds. These flies do not bite, but may get into the eyes, nostrils and ears. Being so small that they pass through ordinary window screening, they can be excluded only with difficulty from dwellings or other buildings in the vicinity of the plant. Their radius of flight is limited to a few hundred feet, but they may be carried by the wind for much longer distances (½ to ¾ mile). *Psychoda* lays its eggs in the filter, and the larvae feed upon the filter slime. The minimum life cycle is 12 days in summer.

Development of filter flies can be checked (1) by flooding the beds for about a day once a week during the periods of greatest prevalence, or (2) by chlorination, either applying chlorine to the sewage in a heavy dose or spraying orthodichlor benzene over the filter. The results of chlorination are still inconclusive. Cultivation of the water springtail (*Achorutes viaticus*) is also said to be effective in reducing fly nuisance. This is probably due to their destruction of the organic slime upon which the larvae of the filter-fly feed. At Fitchburg, Mass., an equilibrium seems to have been established between other insects and flies, and no group gains the ascendancy to such an extent as to become seriously troublesome.

376. Winter Maintenance.—Filters generally are not as efficient in winter as in summer. As in the case of all biologically-activated processes, warm weather is more favorable than cold. In addition there are certain physical obstacles which must be overcome during cold weather. These are especially important in the operation of intermittent sand filters. More care is required in dosing sand beds during the winter than at other seasons. Large doses of sewage must be applied to thaw the frost in the sand or to melt the snow on the surface of the filter. On the other hand, it is essential that the dose be not large enough to freeze to the sand before it can find its way into the filter.

Various methods have been devised for preventing the ice from freezing to the sand. One of the most successful of these is

to furrow the filters for use in winter. The furrows may consist of ridges and depressions about 3 ft. apart on centers and about 10 in. deep. When ice has formed on the sewage, the ridges hold it up as the sewage recedes, leaving a space between the bottom of the furrow and the ice, through which future applications of sewage can flow. This covering of ice also protects the surface of the bed from the extreme cold, preventing serious freezing, and serves to keep succeeding snowfalls out of the sewage.

When sewage is applied to furrowed filters the suspended matter tends to settle to the bottom of the furrows, which afford a much smaller area for the distribution of such matter than the level surface of the bed at other seasons. Where beds are operated at nearly their maximum capacity, especially with sewage containing large quantities of suspended matter, the winter clogging may be serious. It is important, therefore, to take advantage of any opportunity afforded for winter cleaning. It has been found that the mat can be readily raised from the sand if it is barely frozen. If, however, it is frozen hard, the crust will be so thick and the quantity of sand clinging to the mat so great, that cleaning will be impracticable.

377. Efficiency of Sewage Filters.—The degree of purification effected by sewage filters must be studied in connection with the many factors that influence their operation. The following very general figures are therefore presented only with a view to giving the student a basis for comparison.

Percentage reduction of	Intermittent sand filters	Contact beds	Trickling filters
Suspended matter	95–98	85–90[1]	85–90[1]
Organic matter	90–98	60–80	60–80
Bacteria	98–99	50–75	80–90

[1] The effluent from coarse contact beds and from trickling filters must undergo final sedimentation for the removal of suspended matter if a high degree of purification is to be secured.

Intermittent Sand Filters.—When sand filters are doing their best work the effluent from the underdrains will be clear, substantially free from suspended matter and practically odorless. It will remain stable indefinitely, even when allowed to stand in a tightly stoppered bottle in a warm room.

The efficiency of various sand filters in Massachusetts is shown in Table 90. These results are for the most part yearly

TABLE 90.—EFFICIENCY OF SAND FILTERS IN MASSACHUSETTS*

(Results of analyses in p.p.m.)

City or town	Free ammonia			Total albuminoid ammonia			Oxygen consumed			Nitrogen in effluent as		Chlorine	
	Applied sewage	Effluent	Per cent reduced	Applied sewage	Effluent	Per cent reduced	Applied sewage	Effluent	Per cent reduced	Nitrates	Nitrites	Applied sewage	Effluent
Amherst	25.6	7.3	71	8.8	0.55	94	53.3	6.2	88	6.10	0.11	234.1	86.0
Andover	35.0	19.9	64	8.4	1.11	87	61.8	10.4	83	1.20	0.07	61.8	68.5
Brockton	67.7	31.8	53	13.3	1.12	92	128.9	15.3	88	4.07	0.12	122.3	120.3
Clinton	38.4	6.6	83	6.9	0.66	90	69.5	8.0	89	8.91	0.07	59.1	52.7
Concord	21.4	2.5	88	5.6	0.12	98	43.2	1.9	96	5.86	0.02	38.8	34.8
Framingham	61.1	17.4	72	15.0	0.70	95	111.3	9.1	92	2.48	0.12	94.9	84.0
Gardner[1]	54.0	14.6	73	17.6	1.01	94	136.1	8.0	94	19.04	0.19	61.6	63.2
Gardner[2]	43.8	21.6	51	7.4	1.82	76	46.2	13.2	71	9.19	0.23	74.8	67.6
Hopedale	49.0	13.1	73	7.1	1.16	84	44.0	9.2	79	21.30	0.06	58.3	50.0
Hudson	45.2	12.1	73	9.1	1.02	89	95.9	9.9	89	8.32	0.20	460.8	450.0
Leicester	35.5	6.7	81	7.1	0.98	86	56.7	9.5	83	6.07	0.13	54.5	40.6
Marion	12.2	2.0	84	3.3	0.37	89	21.8	3.9	82	4.58	0.02	37.6	36.5
Marlborough	51.8	5.0	90	5.7	0.36	94	44.5	3.8	91	18.47	0.04	98.6	78.6
Milford	39.2	6.9	82	6.1	0.39	92	51.8	5.1	90	15.04	0.09	98.7	97.9
Natick	36.7	11.6	68	8.5	0.57	93	57.8	5.9	90	4.76	0.21	89.0	65.7
Northbridge	37.7	6.5	83	6.5	1.07	84	43.1	7.5	83	9.23	0.08	54.2	43.8
Norwood	40.7	12.3	70	8.4	0.75	91	121.7	11.8	90	1.91	1.56	34.7	298.0
North Attleborough	11.5	0.6	95	1.8	0.12	94	12.8	2.0	84	4.94	0.02	34.1	29.9
Pittsfield	20.4	3.9	81	5.1	0.44	91	36.5	5.2	86	10.31	0.10	52.1	46.9
Southbridge	52.2	19.1	63	7.4	0.87	88	55.5	9.6	83	1.96	0.12	68.2	59.5
Spencer	56.8	5.4	91	12.6	0.38	97	72.3	5.1	93	4.77	0.06	88.3	55.0
Stockbridge	13.3	1.7	87	2.9	0.34	88	18.9	3.6	81	3.02	0.03	20.3	27.9
Westborough	31.4	4.1	87	8.4	0.72	91	49.7	7.0	86	11.52	0.28	82.9	48.8
Worcester	36.4	21.7	40	12.9	1.50	88	126.8	18.2	86	2.20	0.15	132.4	138.3

* Data taken from Massachusetts State Board of Health Report for 1912, pages 371–379. [1] Gardner area. [2] Templeton area.

averages of monthly samples. It will be seen that there is considerable difference in the quality of the sewage applied as well as in the effluents. It is probable that a majority of the analyses of applied sewages represent more accurately the stronger day sewage. Filtration does not affect the quantity of chlorine, so that the chlorine results may be used to determine whether the effluent probably corresponds to the sewage analyzed. It will be seen that there are wide discrepancies, and these should be taken into account when studying the results. In some cases the effluent has doubtless been diluted with ground water. These

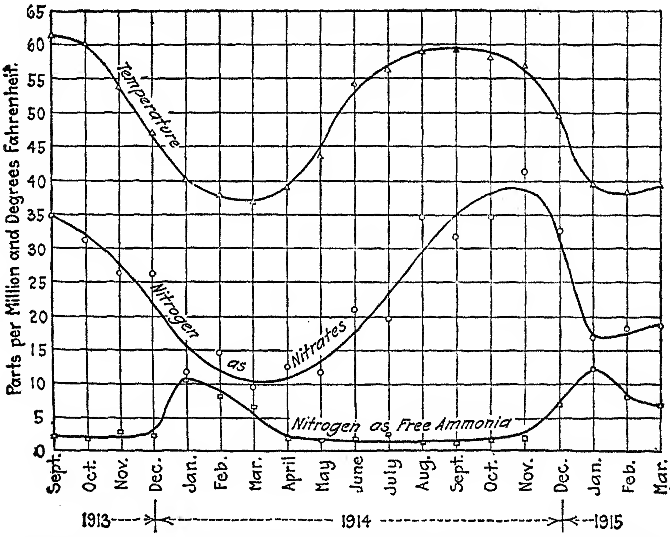


FIG. 184.—Effect of temperature on results of intermittent filtration.

results are not given with a view to showing the relative amount of work done at different plants, as other conditions must be taken into consideration, particularly the character and quantity of sewage applied. These results do show, however, in a general way, the purification accomplished in practice.

The efficiency of sewage filters in a northern climate will not be so great during the winter months as during the remainder of the year (Fig. 184). Bacterial activity is reduced by fall in temperature. The large and irregular doses applied to the filters in order to keep them thawed out and to obtain the best result during the cold weather are not favorable to a high degree of purification.

TABLE 91.—AVERAGE ANALYSES OF INFLUENTS AND EFFLUENTS OF CONTACT BEDS AND TRICKLING FILTERS*
(Results are stated in p.p.m., except as noted)

City	Suspended matter			Settleable solids, 2 hr. (cc. per liter)		Oxygen consumed, 30 min. in boiling water			Alkalinity as $CaCO_3$ by methyl orange			5-day biochemical oxygen demand			Nitrate nitrogen in effluent
	Influent	Effluent	Per cent removal	Influent	Effluent	Influent	Effluent	Per cent removal	Influent	Effluent	Per cent removal	Influent	Effluent	Per cent removal	
Contact Beds															
Alliance	69	17	75	Tr.[1]	Tr.	38	26	32	192	167	13	68	20	71	2.8
Canton	93	41	56	0.3	Tr.	38	18	53	343	354	−3	90	22	76	0.3
Trickling Filters															
Atlanta (Intrenchment)	67	42	34	0.6	Tr.	21	10.0	52	82	48.0	41	23	4.2	82	3.0
(Peachtree)	68	38	44	0.2	Tr.	18	7.0	61	69	35.0	49	20	3.9	81	2.0
Baltimore	107	46	57	0.6	Tr.	36	14.0	61	144	63.0	56	125	14.0	89	3.3
Columbus	79	81	−2	Tr.	Tr.	46	28.0	39	219	192.0	12	134	49.0	70	0.6
Fitchburg	63	52	17	Tr.	Tr.	37	19.0	57	99	7.9	92	95	20.0	79	6.1
Lexington	67	40	40	Tr.	Tr.	22	11.0	50	194	152.0	22	87	16.0[2]	70	
Reading	85	48	44	Tr.	Tr.	31	16.0	48	177	105.0	41	96	20.0	79	4.4
Rochester (Brighton)	38	26	32	Tr.	Tr.	15	9.7	35	189	131.0	31	36	7.1	80	2.8
Average	72	47	34	....	....	28	14.0	50	147	92.0	39	77	17.0	78	

* *Public Health Bull.* 132.
[1] Tr. indicates "trace" (less than 0.1 cc.).
[2] Effluent of secondary sedimentation tanks.

TABLE 92.—OPERATING RESULTS OF TRICKLING FILTERS

(Results are stated in p.p.m., except as noted)

	Brockton, Mass. 1928		Fitchburg, Mass., 1928				Schenectady, N. Y. 1928		Worcester, Mass. 1928	
			Dec. to May		June to Nov.					
	Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent
Nitrogen										
Ammonia	27.4	15.6	11.12	3.34	11.97	2.00	12.7	5.1	14.6	9.6
Albuminoid										
Total	6.67	5.44	2.63	1.50	2.54	1.40	3.43	2.49		
Dissolved	5.60	3.96	1.72	0.98	1.70	0.83	2.03	0.96		
Suspended	1.07	1.48	0.91	0.52	0.84	0.57	1.40	1.53		
Nitrite	0.25	0.45	0.084	0.219	0.028	0.111	....	0.484	....	0.173
Nitrate	0.15	10.15	0.962	6.236	0.437	9.287	......	4.742	....	5.71
Oxygen consumed										
Total	95.4	40.7	88.7	58.8	80.2	50.5				
Dissolved	70.5	26.3	53.9	31.3	45.4	25.4				
Suspended	24.9	14.4	34.8	27.4	34.8	25.1				
Solids										
Total	494	418	287	259	320	300	418	414	569	548
Volatile	237	161	123	113	147	141	....	....	173	145
Dissolved	404	358	239	227	276	266	369	352	465	448
Volatile	169	116	102	97	122	123	....	....	112	107
Suspended	90	60	48	32	44	34	49	62	104	100
Volatile	68	45	21	16	25	18	....	....	61	38
Settling, 1 hour, cc. per liter	....	....	....	....	....	....	....	....	0.77	2.82
Chlorine	77.8	80.2	42.2	41.8	46.0	46.0	38.6	37.2	87.0	87.6
Biochemical oxygen demand	....	....	....	....	....	....	107	35	98.8	20.4
Dissolved oxygen, per cent saturation	6.8	53.8	....	....	....	....	13.8	47.3		
Hydrogen-ion concentration, pH	....	....	....	....	....	....	7.3	7.4		

Contact Beds.—The effluent from fine-grained contact beds is fairly clear and stable. Coarse-grained contact beds, however, unload solids from time to time, but not to the same extent as trickling filters. Final sedimentation is therefore generally required when the average size of ballast exceeds about 1 in. Results of analyses of samples collected during a 10- to 15-day period by the United States Public Health Service in a study of American sewage treatment plants are shown in Table 91.

Trickling Filters.—The trickling filter is capable of converting putrescible settled sewage into highly stable effluent with low B.O.D. During the course of a normal year's operation the suspended solids in the effluent should be about equal in amount to those in the influent. This shows that unloading is complete and that there is no clogging. However, the suspended solids in the influent are very fine and do not settle readily, while those in the effluent are coarse and flocculent, and can be removed by sedimentation in large measure. The volume of suspended matter in the effluent is particularly great during the spring unloading period. At this time masses of sludge worms differing greatly in variety are mixed with the solids.

The studies of filter efficiency conducted by the U. S. Public Health Service are shown in Table 91. Operating data of typical American plants are given in Table 92.

PRINCIPLES OF DESIGN

378. Selection of Filtering Materials.—The filtering material selected for sewage beds affects in one way or another the rate of filtration, the quality of the effluent, the amount of clogging and the cost of construction and operation. For these reasons it must be given careful consideration in the design of the plant. Among the factors to be weighed are (1) size, (2) permanency, (3) roughness, and (4) cost of the material. Since the sand of intermittent filters differs so greatly from the ballast of contact beds and trickling filters the two types of material will be considered separately.

Filter Sand.—Two related measures, first suggested by Allen Hazen,[1] are commonly employed to describe the particle size of filter sand, namely the "effective size" and the "uniformity coefficient." They are determined by passing a known weight of

[1] For a discussion of this subject see Am. W. W. Ass'n: "Water Works Practice," 1925; pp. 634 to 649.

sand through a nest of sieves of wire cloth, with progressively smaller mesh openings,[1] and weighing the amount of sand passing each sieve. The "effective size" is that size, commonly expressed in mm., than which 10 per cent of the sample by weight is smaller; the "uniformity coefficient" is the ratio of the 60-per cent size to the 10-per cent size. Hazen fixed upon the effective size as best representing the relative position of different sands with respect to capillarity and frictional resistance to the passage of water. The uniformity coefficient defines the size range within which 50 per cent of the particles lie, and reflects to a certain extent the porosity of the sand: generally speaking, the more uniform a material the greater is its pore space. On the basis of effective size the capillary rise of water (1) and the frictional resistance offered by the sand (2) were represented by Hazen in the following expressions

$$H = \frac{1.5}{d^2} \tag{1}$$

$$v = cd^2\frac{h}{l}\left(\frac{T + 10}{60}\right) \tag{2}$$

where H = height in mm. to which water will be lifted by capillarity in sufficient quantity to prevent circulation of air.

d = effective size in mm.

v = velocity in meters per day of a solid column of water of the same cross-section as that of the sand (approximately also rate of filtration in million gal. per acre daily).[2]

h = head of water causing motion.

l = thickness of sand $\left(\frac{h}{l}\right.$ = loss of head in feet per foot of sand).

T = temperature in degrees Fahrenheit $\left(\frac{T+10}{60}\right.$ = viscosity factor, see Section 311).

c = a coefficient varying with the compactness of the sand and, in the case of intermittent sand filters, with the displacement of the air in the filter. In water filtration, values of 500 to 1,200 are generally observed.

[1] Ratios of sieve openings of $1:\sqrt{2}$ or $1:\sqrt[4]{2}$ are common.

[2] Assuming a porosity of 40 per cent, the actual average velocity of flow through the pore spaces will be 2½ times that given by equation (2).

These formulas are here presented with a view to calling the student's attention to the hydraulic characteristics of sand, rather than to their use in design of sand beds. The effective size of sand most commonly associated with intermittent filtration of sewage varies from 0.1 mm. to 0.5 mm. and the uniformity coefficient is usually less than 15.

Clean quartz sand is best suited for use in intermittent sand filters. It should not be too coarse, which would permit too rapid filtration, insufficient contact and deep penetration of fine solids, or too fine, so as to limit the water load too greatly and decrease aeration by too long retention of the sewage and capillary saturation of the sand.

Where suitable sand or gravel is available in place, a filter bed can be constructed by grading its surface to receive sewage. Loam and silt have a tendency to hold water by capillarity and to reduce aeration, unless the filter is operated at a very low rate. Clay, cementitious sand and other relatively impervious materials are useless for intermittent filters.

Loam and subsoil must be removed if filters are to be operated at a reasonable rate. The extent of removal is usually determined by the cost of the work. Large stones and roots of trees, together with the fine earth about them, must be cleared away. The aim should be to obtain a bed of uniform permeability, so that differences in hydraulic resistance will not cause overloading at places where the sewage passes through the sand most freely.

Ballast.—Many different materials have been used as ballast in contact beds and trickling filters: crushed stone, pebbles, blast furnace slag, cinders, coke, coal, broken brick, waste from potteries, wooden laths, brush (bundles of faggots), and corncobs. In the United States crushed stone seems to be preferred to other materials, except in very small installations where laths laid in a crisscross pattern have been employed. Crushed stone does not disintegrate so rapidly as do other materials under the influence of sun and air, freezing and thawing, and wetting and drying. It is not broken down by the weight of overlying material, and is not subject to organic decay. Being heavier than water it does not alternately rise and settle, as may coke in contact beds. Trap rock, granitic rock and limestone are employed commonly. Of these, trap rock appears to be a particularly satisfactory material. While preferences are well established, it may be more economical to use cheap materials

like cinders, even though they must be renewed occasionally, than to build of more permanent but considerably more costly material.

Standard methods of testing and selecting filter materials are being evolved.[1] In general the tests follow those employed in connection with highway materials, and include specific gravity, absorption of water, wear or resistance to abrasion, toughness or resistance to fracture, soundness or ability to withstand conditions of service without disintegration, and chemical analysis.

The greater the surface area of the filtering material the greater is the opportunity for contact between the sewage and bacterial jelly covering the filtering medium. The extent of surface varies directly with roughness and inversely with the size of the ballast. Surface contour, however, is rapidly suppressed by the growths which cover the ballast. Roughness is helpful in holding the gelatinous film, but tends to retard the unloading of trickling filters, and may induce clogging. Very smooth materials such as water-worn pebbles, on the other hand, afford insufficient hold for the zoöglœal growths. Size affects the opportunity for oxidation also by controlling, together with depth, the time of contact in the case of trickling filters.

The time of contact or passage through filters of different depths and sizes of ballast which was observed at the Lawrence Experiment Station is given in Table 93, but this time, it should

TABLE 93.—TIME OF PASSAGE OF SEWAGE THROUGH EXPERIMENTAL TRICKLING FILTERS, LAWRENCE, MASS.

(Report of the Massachusetts State Board of Health, p. 1908; p. 386)

Material	Size of material, in.	Depth		Ave. daily rate[1]		Time of passage of sewage, hours	Nitrates, p.p.m.
		ft.	in.	Gal. per acre	Gal. per cu. yd.		
Broken stone.....	¼–1	10	0	1,150,000	71.3	3	29.5
Broken stone.....	¼–1	10	0	1,938,000	120.1	2	20.5
Broken stone.....	¼–1	8	0	963,400	74.6	2	21.2
Broken stone.....	¼–1	5	0	925,500	114.7	2	5.2
Coarse clinker....	¾–1¾	5	9	885,000	95.4	6	12.4
Coarse clinker....	¾–1¾	3	10	949,000	153.5	1	7.3
Fine clinker......	¼– ¾	5	9	896,500	96.6	14	22.0
Fine clinker......	¼– ¾	3	10	908,900	147.1	6	5.0

* Six days a week.

[1] *Proc.*, A. S. C. E., 1928; **54**, 243 (Society Affairs).

be understood, relates only to the conditions of those particular experiments. Dunbar found[1] that if a quantity of sewage equal to the water-retaining capacity of a filter 3 ft. deep is applied to such a bed, 20 to 30 min. will elapse before that sewage is discharged from the bottom.

Numerous experiments with trickling filters of different sizes of coal, gravel and clinker from refuse destructors were carried out by William Clifford, who concluded from the results he obtained that the time of percolation through clean filtering material varies inversely as the rate of sprinkling and directly as the amount of water taking part in the general water movement through the bed, the amount of water in motion being generally represented by the interstitial water.

The Royal Commission on Sewage Disposal[2] has indicated British experience with the size of ballast and corresponding permissible loadings for contact beds to be as follows:

Type of sewage	Suspended solids, p.p.m.	Size of ballast
Crude	400	3 in., upward
Septic tank effluent	80–100	3/8–5/8 in.
Chemical precipitation effluent	10–30	1/4 in.

Another point to be considered in connection with the size of the contact material is the effect of the size on the clogging of the bed. Beds constructed of materials of the sizes recommended by the Royal Commission have often become so clogged that these materials had to be removed, washed and replaced after 3 to 5 years' use. This is an expensive operation. There is some evidence that if contact material not smaller than 1 in. in size is used and the management of the beds is good, serious clogging of the beds can be avoided, but it is questionable if the average results obtained with such coarse beds will be as good as those obtained with the finer beds, unless a much greater quantity of contact material is used so as to obtain adequate area of gelatinous surface to effect the desired changes in the

[1] "Sewage Treatment," p. 140. The water-retaining capacity of a soil is defined as the volume of water which remains in the soil after drying the soil, filling it with water and allowing the excess water to drain away.

[2] *Fifth Report*, p. 68.

applied liquid. Furthermore, with such coarse beds, the underdrainage must receive greater attention than is necessary with beds of finer material and may require more expensive construction, and the effluent may contain such a large quantity of suspended organic matter at times that settling basins should be provided for its removal, as is the case with the effluents from trickling filters. Some contact beds are covered to a depth of 6 in. with white pebbles or other clean material which is not dosed with sewage and presents a pleasing appearance. Where double contact is employed the primary bed is often constructed of coarse material (3 or 4 in.), and the secondary bed of fine material.

In the case of trickling filters, American practice shows a tendency to use stone from 1 to 2½ in. in size. As small material should be used as is consistent with abundant air supply and freedom to unload from time to time the solids which have accumulated in the filter. Unloading is apparently facilitated by having the filtering material fairly uniform in size, for if the voids between the large stones are filled with finer stones the sewage solids may be retained indefinitely in the bed and cause clogging. In practice difficulty is often experienced in securing stone within the specified sizes, and unless fine material is definitely excluded at the crusher, the stone should be rescreened before it is placed in the filter.

The character of the applied sewage must be considered in selecting the size of filtering medium. The more thoroughly the suspended matter is removed by preliminary treatment, the finer the filtering medium it is safe to choose. Generally sewage is fairly well settled before it is applied to trickling filters in order to reduce the danger of clogging to a minimum and to avoid placing upon the filter a burden of work which can be performed by less expensive structures like tanks.

In trickling filters in which the surface layers are particularly exposed to weathering, as is the case in the northern United States, less resistant but locally cheaper ballast such as limestone may be employed for the lower layers, while materials such as trap rock may be used for the upper ones.

379. Choice of Bed Depth.—The relation between depth of bed and permissible loading, as shown in Section 370, is important except, perhaps, in the case of sand filters. Proceeding to the design of filters another factor must be taken into consideration,

namely the head available. This may fix the limits of design. Topography, too, is of considerable influence.

Intermittent Sand Filters.—Although very little added improvement is obtained by making sand filters deep, it has been found advantageous practically to have 3 or 4 ft. of sand above the underdrains in order to prevent the sewage from breaking through and reaching the collectors in an inadequately oxidized state. The greater depth has a steadying effect upon the bed efficiency, requires a less elaborate system of underdrains (Section 382), and during the life of the bed permits removing more sand in the course of the cleaning operations. To ensure unsaturated sand near the surface, filters must be deep enough also to offset the capillary rise of water. Fine sand with effective size 0.1 m.m., for example, will raise water by capillarity $\frac{1.5 \times 12}{0.1^2 \times 302.4} = 6$ in. (Section 378) more or less. On the other hand, beds should not be made so deep that ventilation is impaired. If anaerobic conditions are established in the bottom layers the effluent is deteriorated in quality and, in the absence of dissolved oxygen, iron is taken into solution and growths of iron bacteria (*Crenothrix polyspora*) may clog the drains.

Clark states in the 1908 report of the Massachusetts State Board of Health, p. 302, that, other things being equal, filters of greater depth give effluents of a higher degree of purification than those of less depth. With coarse sands, 0.25 mm. effective size or larger, 4 or 5 ft. in depth of bed was found desirable. A filter 10 ft. deep gave a somewhat better effluent, but not markedly so. Shallow filters (2 ft.) of coarse sand gave fair results when operated at low rates. Where intermittent filters are provided to follow contact beds or other oxidation processes (Section 368) they are sometimes given depths of only 2 to 3 ft. Underdrains must be more closely spaced in shallow beds.

Contact Beds.—Method of drainage, maintenance of beds in good condition, and restriction of area occupied are the chief factors to be considered, besides head available, in choosing the depth of contact filters. Beds are commonly made 4 to 6 ft. deep.

Ordinarily these beds have pipe underdrains laid in large stones, making an underdrainage system at least 6 in. deep. The sewage which remains in this underdrainage system is not so well treated as that in the main body of the bed, where the smaller

size of the stones affords a much greater area of gelatinous film per cubic yard of material. There is little to be said in favor of such underdrainage, and better drainage undoubtedly can be afforded by a floor system similar to those now used with trickling filters. Experience with British contact beds, particularly those at Manchester, has shown that loss of capacity often was due more to imperfect drainage than to an accumulation of solids. In any case, it seems undesirable to have this least efficient part of the bed more than one-fifth of the whole depth, and for this reason the Royal Commission suggested 2½ ft. as the minimum allowable depth of bed.

Deep beds place a considerable weight on the material in the lower part, which may lead to its disintegration. Furthermore, when washing becomes necessary, it is much more difficult to remove and handle the material from deep than from shallow beds and the danger of breaking it up is increased. For this reason the Royal Commission suggested that 6 ft. was probably the maximum limit of depth.

Trickling Filters.—Trickling filters are commonly from 5 to 10 ft. deep, with 6 to 8 ft. considered a satisfactory mean. The depth of a trickling filter should be limited by the requirements of proper ventilation, for if the oxygen of the air passing through the filter is used up before the bottom is reached the lower part of the filter will be ineffective. For treating unusually strong sewages or industrial wastes having a marked avidity for oxygen, such filters should not be so deep as when ordinary domestic sewage is to be treated. With abnormally dilute sewage a deep filter requires the application of a large volume of sewage per unit area if the capacity of the filter is to be based upon a normal per capita loading. This consideration may lead to the adoption of a moderate depth as most economical. There will be a greater tendency toward surface clogging and consequent obstruction of the circulation of air in deep beds on account of the greater quantity of sewage applied per unit of superficial area. This condition is governed largely by the character and size of the filtering medium and the efficiency of preliminary clarification. Organic growths have appeared at times on some filters, more or less clogging the surface layer. Uneven distribution will more quickly result in clogging over-dosed areas in the case of deep beds because of the higher rate of application per unit of area. If portions of the filter area must be rested

from time to time on account of surface clogging, a larger proportion of the filter will necessarily be out of use in the case of deep beds.

In most cases where sufficient head is available, the cost of trickling filters per unit of volume decreases with the increase in depth, owing principally to the reduced cost of floor, underdrainage and distribution with deeper beds. If the quantity of sewage which can be purified satisfactorily by trickling filters is proportional to the depth, the deeper filters will be the more economical. Estimates made by one of the authors, based on the contract prices for the trickling filters at Fitchburg, Mass., showed that the cost per effective cubic yard of filtering medium was $4.54 in the case of 6-ft. depth, $4.20 with 7-ft. depth, $3.96 with 8-ft. and $3.60 with 10-ft. depth.

380. Size, Shape and Grouping of Beds.—The size, shape and grouping of sewage filters is commonly dictated by considerations of (1) topography, (2) arrangements for distributing the sewage over the beds and collecting the effluent in underdrains, (3) cleaning, resting or repair of beds, (4) economy of large units versus small units, and (5) storage of sewage where intermittency of operation requires.

Intermittent Sand Filtration.—In the larger intermittent sand filtration plants, beds having areas from ¾ to 1 acre generally have proved most desirable. In smaller plants the size may be much less to avoid throwing a large proportion of the area out of use when cleaning beds, and to facilitate dosing without storing the sewage too long.

Except in very small plants, intermittent filter beds are rectangular in shape when the topography permits. The cost of embankments, which are usually made of the loam stripped from the beds, is so small that it is not often a factor to be considered in the determination of the shape.

The underdrainage system and the means for distribution of sewage over the bed are usually of most importance in determining the shape. With square beds it is practicable to flood them satisfactorily from the corners, even when they are of large size, but the underdrainage is likely to cost more than when long beds are used of such width that two can be drained by laterals running to a main drain laid in the embankment between the beds. The distribution of sewage over long beds is not so

uniform as it is over square beds unless troughs are used, which operators dislike.

As a general thing, several arrangements of beds, distributing conduits, drains and roads are practicable, and some preliminary studies will be needed to determine which is best. The cost of the whole installation rather than that of one or two beds should be the deciding factor, since main drains or main carriers may prove unexpectedly expensive if judged by an examination of the needs of one or two beds.

Contact Beds.—The size of the units is more important with contact beds than with most other methods of treatment. Each unit should be small enough so that under normal conditions of sewage flow the time required for filling shall not be unduly long. Large units are inadvisable also on account of the long time required for drainage. At Manchester, England, with a sewage flow of about 36,000,000 gal. daily, the individual beds were made ½ acre in area. At Plainfield, N. J., and Mansfield, Ohio, the units were made 0.22 and 0.25 acre respectively.

The beds at Mansfield, Ohio, designed by Barbour, are laid out in the form of a circle, each of the ¼-acre beds forming a sector separated from the next by earth embankments.

Trickling Filters.—The shape of the filters will be controlled in part by the topography of their site and to a large extent by the distribution scheme used (see Section 381). The size of circular filters with mechanical distributors is limited by the economical size of the distributing device, and is influenced by the fact that it is less expensive to build a few large beds than many small ones. For filters using mechanical distributors more than one unit should be provided, so that in case repairs become necessary the entire plant need not be shut down. The Royal Commission on Sewage Disposal recommended not less than three units for each plant. There is a decided advantage in having small units, for they permit resting portions of the filter at intervals. A filter fed by fixed nozzles may be divided into as small units as desired. At Columbus, the 10 acres of trickling filters are divided into six units, each an equilateral triangle and all arranged about a central point. This arrangement is advantageous for distribution but not for some forms of underdrainage. It has proved unsatisfactory to have the administration building in the center of the plant, owing to obnoxious odors and flies.

381. Distribution of Sewage.—The means provided for distributing sewage over the beds should be such as to produce as nearly uniform loading of the filter as is obtainable and warranted by considerations of economy. The higher the rate of dosing the more important does uniformity of distribution become. It follows that the distribution systems developed for trickling filters are more elaborate than those employed in the older treatment methods.

Intermittent Sand Filters.—The distributors which carry sewage to the individual beds of intermittent sand-filtration plants are commonly pipe sewers laid in embankments between the beds. Banks which serve as driveways are generally 8 ft. wide with side slopes of 1:1 or, preferably, 1 vertical on 1½ horizontal. Partition banks are seldom over 2 ft. wide at the top. Gravity sewers of sewer pipe or pressure sewers of cast iron pipe are laid with a cover of about 2 ft. This depth is sufficient to protect sewer pipe against breakage. It is also adequate to prevent freezing, as the sewage is comparatively warm. The height of the embankments above the beds is determined by topography and quantity of soil to be disposed of and by the hydraulic gradient required. The size of the main distributors must be such as to permit dosing beds at the proper rate. A rate of 1 cu. ft. per second for each 5,000 sq. ft. of area dosed is common. A single bed or a group of beds may be dosed at a time, depending upon the method of control adopted.

The sewage may be distributed over the beds by the following methods:

1. Single graduated troughs running nearly the full length of the beds, as shown in Figs. 180 and 185.
2. Radiating or arterial troughs, used particularly for irregular-shaped beds, or where more even distribution is to be secured, as with filters dosed at a high rate, as are those following contact beds.
3. Quarter-point distribution, which requires the discharge of sewage at the two quarter points on the long sides of the bed.
4. Corner distribution, in which the sewage is discharged upon the bed at or near one or more of its four corners.
5. An outlet, located at the mid-point of one or both of the long sides of the bed, and designed to distribute sewage radially through a number of small holes in a semicircular concrete curb. This method, which was used for a time at Clinton, Mass., has been abandoned there as unsatisfactory.

Where troughs are not used, it is desirable to provide headwalls and a paved area at the point of discharge of the sewage, to prevent erosion of the surface of the bed.

Filter beds are generally graded substantially level. There is no advantage in sloping the surface under ordinary conditions, for if the discharge of sewage upon the bed be rapid, satisfactory distribution will be obtained with a level bed.

The best method of controlling the distribution of the sewage will depend upon the size of the works and the topographical conditions. Where hand control is employed in large works sewage may be applied simultaneously to a group of beds by

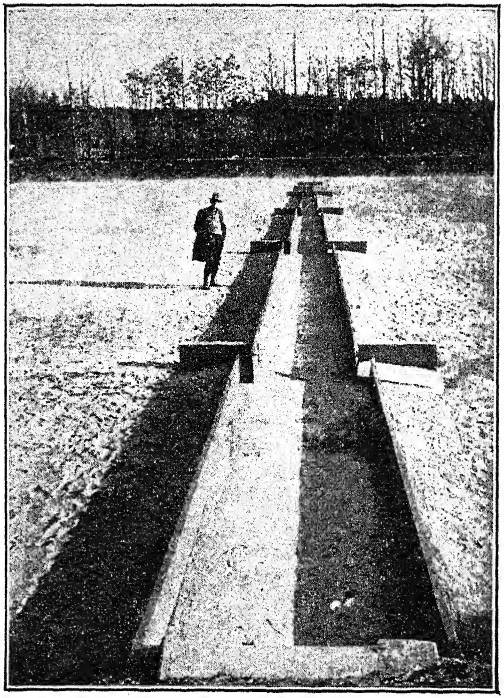


Fig. 185.—Intermittent sand filter with sewage distributor, Marlborough, Mass.

master valves at central points, each outlet for the sewage on a bed being controlled by a gate on the lateral distribution pipe. In small filter plants, the distribution is usually controlled by gates at the individual beds.

The discharge of sewage at these distribution points should be controlled by means of shear or sluice gates in a manhole, from which the distributors branch. The simple shear gate is the cheaper and has been found on the whole to give reasonably satisfactory service.

Automatic distribution control will be dealt with in Section 384. Sub-surface distribution of sewage has been referred to in Section 305. It is employed only for small installations where it is desired to keep the sewage out of sight and odors must be avoided.

Contact Beds.—The chief aim in the design of the distribution system for contact beds must be to fill the bed without disturbing the deposits in the filter and the bacterial films on the contact surfaces. Small beds are sometimes filled from one or more openings in the sides or corners of the bed; this is known as lateral filling. Larger beds are more commonly provided with troughs running over the surface, or with pipes laid about a foot beneath the surface. The distributing system may radiate from one corner or may consist of a main carrier from which laterals are branched off like the arteries of a leaf. Troughs are generally provided with side outlets, and pipes are usually laid with open joints. Sub-surface distribution avoids unsightliness of the bed surface (see Section 378) and sewage odors. Filling from the bottom through the underdrains has been employed, but is open to the objection that the sewage filling the underdrains is not subjected to contact treatment. Filling rates in excess of 0.1 to 0.2 ft. per minute of vertical rise should generally be avoided. At Manchester, England, open-bottomed troughs partly filled with fine material, through which the sewage is screened, have been employed (Fig. 191). In some American plants clogging has been reduced by replacing 6 to 12 in. of ballast at one corner of the bed with fine cinders and constructing a low bank of cinders around this area. Sewage is applied at this corner and is filtered through the cinders before filling the bed. The clogged cinders are raked off from time to time. For automatic control of contact beds see Section 384.

Trickling Filters.—A number of different means have been devised for securing uniform distribution of sewage over trickling filters. Some of these are outlined in the following schedule:

1. Moving distributors.
   (*a*) Revolving (Fig. 186).
   (*b*) Traveling the length of the bed (Fig. 187).
   (*c*) Tipping trays (Fig. 188).
2. Stationary distributors.
   (*a*) Splash plates.
   (*b*) Spraying nozzles (Fig. 189).
   (*c*) Filtering layer of fine material (Dunbar).

Revolving and traveling distributors have been used extensively in Great Britain and its dominions, and in some places in the United States.

Revolving distributors commonly consist of two or four horizontal arms revolving about a central post and dosing circular filters. Traveling distributors usually take the form of a low truss spanning a rectangular bed and moving back and forth on its side walls while distributing liquid over the bed in various ways, according to the type of the machine.

The revolving distributor used for dosing two filters 30 ft. in diameter, shown in Fig. 186, is typical of small British apparatus. There is a revolving distributor 130 ft. in diameter at Malvern, and others from 70 to 120 ft. in diameter have been

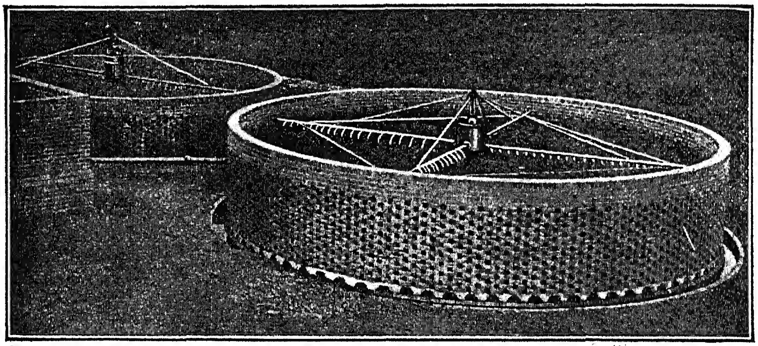


Fig. 186.—Revolving distributor.

constructed for different places, but most installations probably have been under 70 ft. In the apparatus shown in Fig. 186 the weight is carried by a ball bearing at the top of the vertical standard. In some other makes the weights of the moving parts are carried by floats. The head lost in these revolving distributors is small.

An entirely different principle is employed in the water-wheel type of distributor, which seems to be favored in Great Britain for dosing filters of fine material where particularly uniform distribution is desired. These distributors can be constructed to revolve or to travel back and forth as shown in Fig. 187. Either type is driven by the fall of liquid through a height of about 18 in. With the rotating type, the liquid is supplied through a hollow standard in the center of the bed, while in the traveling type it is siphoned out of a channel running along one

side of the bed or between each pair of beds. The liquid passes through a feed tube running the whole length of the distributor. This tube has apertures through which the liquid drops into the buckets of a long waterwheel. The feeding arrangements on the distributor are reversed at each end of the bed by a buffer attached to the masonry, and as soon as reversal occurs the apparatus begins its return trip. The beds shown in the illustration are 92 ft. long, 14 ft. wide and 3 ft. deep.

Tipping trays (Fig. 188) are used only for small installations, notably in connection with lath filters, where they require triangular boards to assist in obtaining uniformity of distribution.

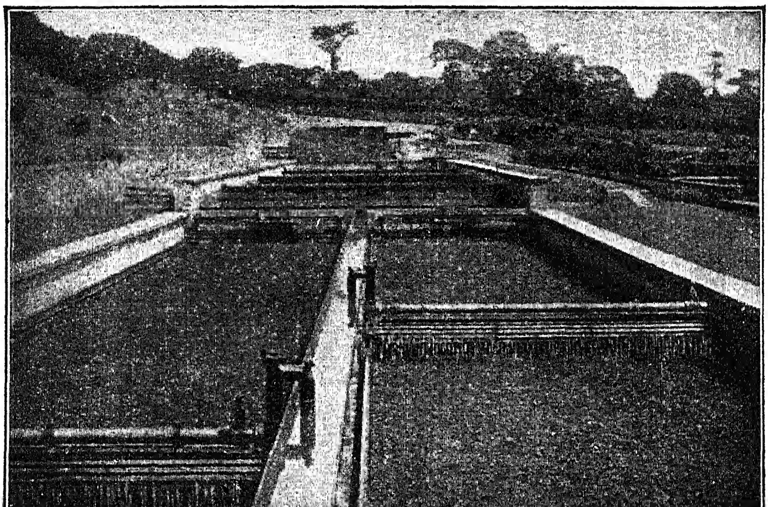


Fig. 187.—Traveling distributor of water-wheel type (Fiddian).

Objections raised to the use of moving distributors for larger installations are (1) troubles with interruptions due to wear of the moving parts, (2) deposition of solids in troughs feeding traveling distributors, (3) freezing troubles in cold climates and (4) clogging of small holes or openings discharging sewage over the bed.

Splash plates commonly receive a stream of sewage from openings in the bottoms of troughs or pipes. The plate consists of a concave metal disk placed above the bed surface, upon which the sewage falls and from which it rebounds upward and outward in the form of spray. The use of splash plates is open to the objec-

tion of clogging of the supply openings and freezing of the exposed distribution system.

Spray nozzles for trickling filters originated at Salford, England, and are employed almost exclusively for dosing trickling filters

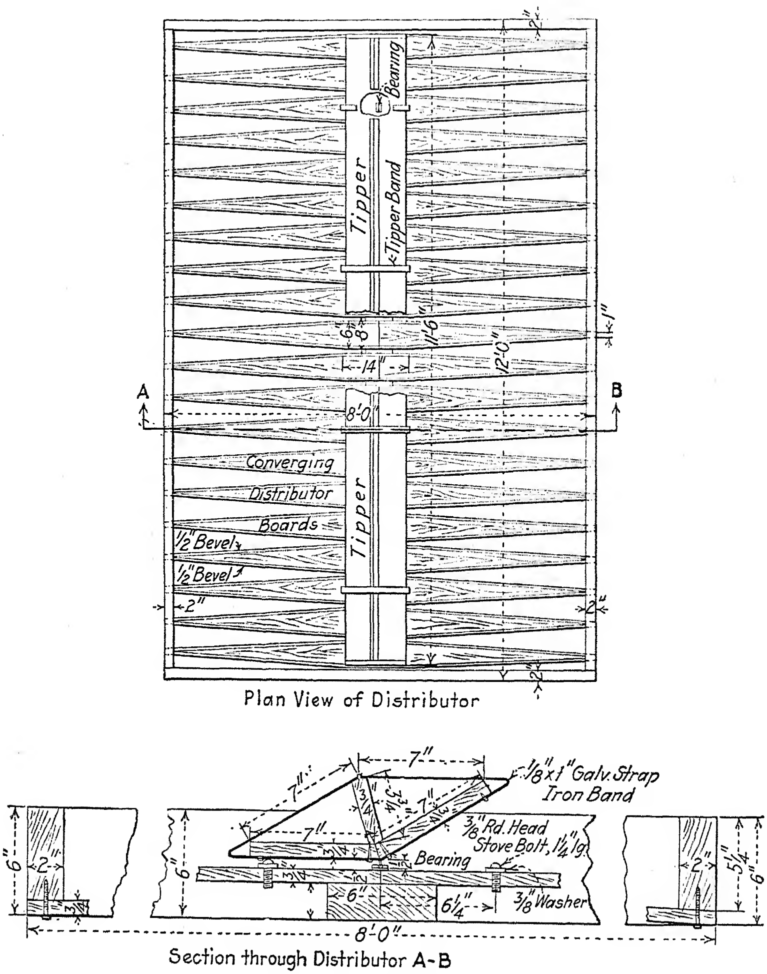


FIG. 188.—Tipping tray distributor.

in the United States. Such nozzles are now made in several patterns, some throwing a spray which covers an approximately square area, and others dosing a circular area. A typical circular-spray nozzle is shown in Fig. 189*a*, illustrating the

nozzle developed at Worcester, Mass. It has at the base of the spraying cone a rim or lip, the function of which is to break up the sewage into a fine spray and spread it over a wide area. The spindle has a locking device permitting its ready removal when cleaning the orifice. A typical square spray nozzle, designed by Taylor, is shown in Fig. 189*b*. It has an orifice 1 in. in diameter, through which a spindle passes, carrying a four-lobed spreading cone, which is intended to throw the desired square-covering spray. The distributing cones of such nozzles must be kept in a definite position in order to spray contiguous square areas. These

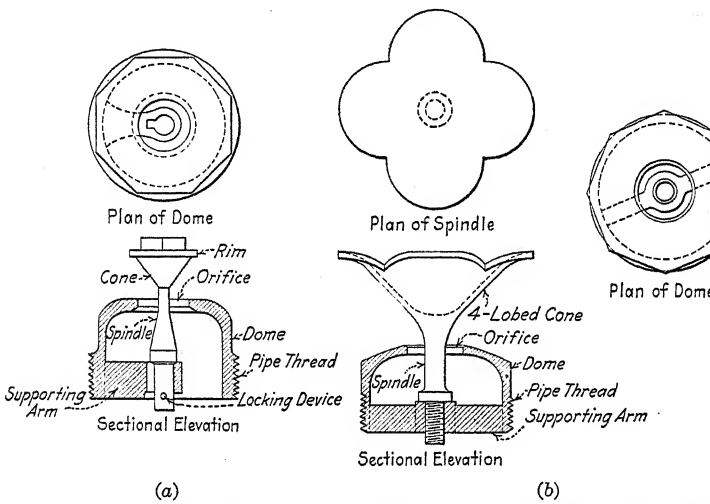


FIG. 189.—(*a*), Worcester type of nozzle; (*b*), Taylor square-spray nozzle.

nozzles are designed to concentrate the spray in a narrow zone and their success in use depends upon rapidly varying the head of the applied liquid. Taylor nozzles also are made to spray hexagonal and circular areas.

The design of a nozzle should be such as to minimize clogging and permit rapid cleaning. Nozzles having small orifices generally produce a relatively fine spray, somewhat greater absorption by the sewage of atmospheric oxygen, and a more uniform distribution, but the cost of keeping them clear may be a substantial item. If the orifice is so large that the sewage is delivered to the bed in a sheet, the streaming effect may prevent efficient oxidation. With a fine spray more odor and a greater

reduction in the temperature of the sewage during cold weather are to be expected than with a coarse spray.

Other conditions being equal, the nozzle which sprays the greatest area under a given head is the most economical. If this result is secured by a concentration of spray in a narrow ring, satisfactory distribution will depend upon a rapid variation in head during the discharge. The efficiency of practically all fixed nozzles may be greatly improved by a variation in head, because a comparatively large proportion of the discharge is concentrated upon a relatively narrow ring of the filter area if the head is constant. Methods of causing such variations are explained later in this chapter. The hydraulics of spray nozzles will receive further attention in Section 385, and methods of automatic dosage control are outlined in Section 384.

Mains for distributing sewage to fixed nozzles may be of concrete or vitrified pipe encased in concrete, but the smaller sizes are preferably of cast-iron pipe with lead or composition joints. The pipes should be large enough to avoid undue friction and consequent loss of head at the nozzles. Economy of construction requires the lateral distributors to be relatively short to avoid the use of large pipe.

The distributing laterals may be laid just above the filter floor, as at Columbus; nearer the surface, as at Baltimore; or at the surface, as at Fitchburg. They may rest on the filtering material with no other foundation, but if it should become necessary to remove the filtering material, the pipes would then have to be supported or temporarily removed. Where the splash plate or gravity nozzle is used, the distributing troughs or pipes are necessarily supported at an elevation above the surface of the bed. It is advisable in all cases to provide ample facilities for flushing and draining the distribution system.

In certain cases fractures have occurred in lateral distributing pipes owing to excessive pressures. If these pipes are laid at the bottom of the filter, repairs upon them will entail excavation of filtering material, which will add greatly to the cost of the repairs. Placing the laterals at or near the surface makes them more accessible, but large pipes placed at the surface are likely to affect the uniformity of distribution somewhat and clogging is more likely to take place about the pipes than on the remainder of the area. In this case also, the sewage in the distributing pipes is exposed to low temperatures of a cold climate to a

greater extent than when these pipes are below the surface of the bed.

If the lateral distributors are at the surface of the filter, as at Fitchburg, the nozzles may be attached directly to the distributing pipes. Nozzles are generally provided with 2-in. pipe-thread connections, which cannot be tapped into cast-iron pipes smaller than 6 in. Screwed pipe and fittings are therefore generally employed below 6 in. size. If the lateral distributors are placed beneath the surface, riser pipes will be necessary. The most common form of riser is cast-iron pipe jointed with bitumen or some other flexible-joint material. Screw pipe has also been employed. The riser should be large enough to minimize friction, due allowance being made for a considerable deposit or growth on the inside of the pipe. Risers are commonly 3 in. in diameter. Long risers are more likely to become bent or broken during construction than are shorter ones. The elevation of the top of the riser above the filter should ordinarily not exceed 6 in., although certain types of nozzles afford better distribution when the nozzle is placed 12 in. or more above the filter surface. In cold climates the exposed portion of the riser is likely to cause trouble from freezing during the resting periods of the dosing cycle. Sloping the bed to equalize the heads on the nozzles, while feasible, results in a slow discharge or "dribble" of sewage from the nozzles at the lower elevations. This is not desirable.

Dunbar, at Hamburg, secured an even distribution of sewage by using small-sized ballast at the surface of the filter. The beds are dosed by means of a central trough and sewage percolates into the filtering material. Dosing is continued until the surface of the bed clogs; the bed is then thrown out of service for rest and surface cleaning. This type of filter has found only limited application.

382. Underdrainage.—Underdrains, as is true of distribution systems, are necessarily made more elaborate as the volume of sewage treated per unit area becomes greater. In the case of intermittent sand filters constructed in localities where deep deposits of sand and gravel occur, underdrains may not be necessary. In the case of trickling filters, on the other hand, suitable design of underdrainage facilities becomes of relatively great moment.

Intermittent Sand Filters.—Vitrified sewer pipe makes the most satisfactory underdrains for intermittent sand filters, because

it is durable and easily cleaned. It should be laid true to line and grade. The bottom of the bed, when constructed artificially, should be level or slope toward the underdrains. With artificial beds, it is desirable to lay the underdrains in trenches below the bottom of the sand, so as to make the entire depth of sand effective for filtration and keep the drains well below the surface. If possible the drains should have a free outlet, but submerged outlets will operate satisfactorily and sometimes trapped outlets are advantageous. In laying the pipe, the spigot end should be about ⅜ in. from the shoulder of the socket to allow the water to flow readily into the pipe. The authors break off the socket on the upper part of the pipe (Fig. 190), leaving the lower part of the socket to keep the pipe in alignment. The drain is then

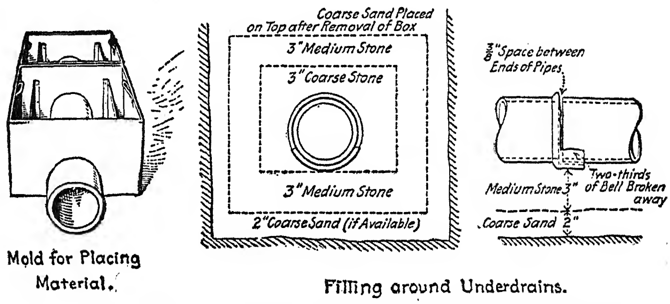


FIG. 190.—Method of constructing underdrains of sand filters.

surrounded with screened gravel, with the coarsest grade next to the pipe. Two or three grades of gravel are used, obtained by passing it through at least two screens, the first having about 1-in. mesh and the second about ¼-in. mesh. It is desirable to discard stones larger than about 2½ in. The coarse and medium grades of gravel are placed in 3-in. layers and unless the sand of the bed is very coarse, a third layer of the finest size is required. The layers of gravel should surround the pipe and not merely cover it. The box shown in Fig. 190 is helpful in placing the gravel rapidly and accurately.

Underdrains are usually laid from 3 to 4 ft. deep at the upper end, on flat slopes with about 6 in. in 100 ft. as a minimum. The usual spacing of the underdrains is about 40 ft., but if the sand has an effective size of 0.08 to 0.12 mm., the spacing should be about 30 ft. The minimum size for an underdrain is 4 in. Beds used for the final treatment of sewage subjected to other oxidation

processes are sometimes provided with drains at shorter intervals, say 10 to 15 ft.

The underdrainage pipe should be connected with a main arterial system of underdrains, which may be laid either with tight or with open joints, the size of the pipe depending upon the quantity of effluent to be handled, the minimum diameter usually being 8 in. The outfall into the stream or open channel may be provided advantageously with a headwall of masonry. If the lines of the underdrains themselves are long, manholes may be desirable at convenient intervals. Provision should be made for sampling the effluent from each bed in small plants and each group of beds in larger ones. Porous concrete pipe has been employed instead of tile pipe for the laterals of some plants. Results of their operating efficiency are not obtainable as yet.

Underdrainage by blind drains formed of large stones is undesirable, because the drains cannot be cleaned without digging down to them and because they do not afford a means for the effluent to flow off rapidly.

Contact Beds.—Some of the first British contact beds were constructed merely in excavations in practically impervious earth, the bottoms and sides being unlined. Experience with them showed a general tendency of earth to work into the contact material and for the underdrains to become distorted by unequal settlement. Where earth embankments were used to separate beds the leakage through them was occasionally so great that adjoining beds had to be filled simultaneously. On the other hand, contact beds at Mansfield, Ohio, constructed with an earth bottom into which hard cinders were rolled to secure as great density as possible and with earth embankments, which went into service in 1902, were found in 1914 to have a total leakage of 5 per cent, mostly through a revolving valve controlling the discharge of the underdrains. In the United States, however, concrete has been employed generally for the walls and bottoms of contact beds.

Fig. 191 shows the general arrangement of the underdrainage and distribution systems of a typical bed used at Manchester, England. The bottoms of the beds are concrete, in which the underdrain channels are formed. These channels are covered with perforated stoneware slabs laid flush with the surface of the bottom. The bottom was graded in ridges with flat slopes to the underdrains. At Mansfield, Ohio, the beds designed by Barbour are underdrained by open-joint tile laid in depressions about 6 in.

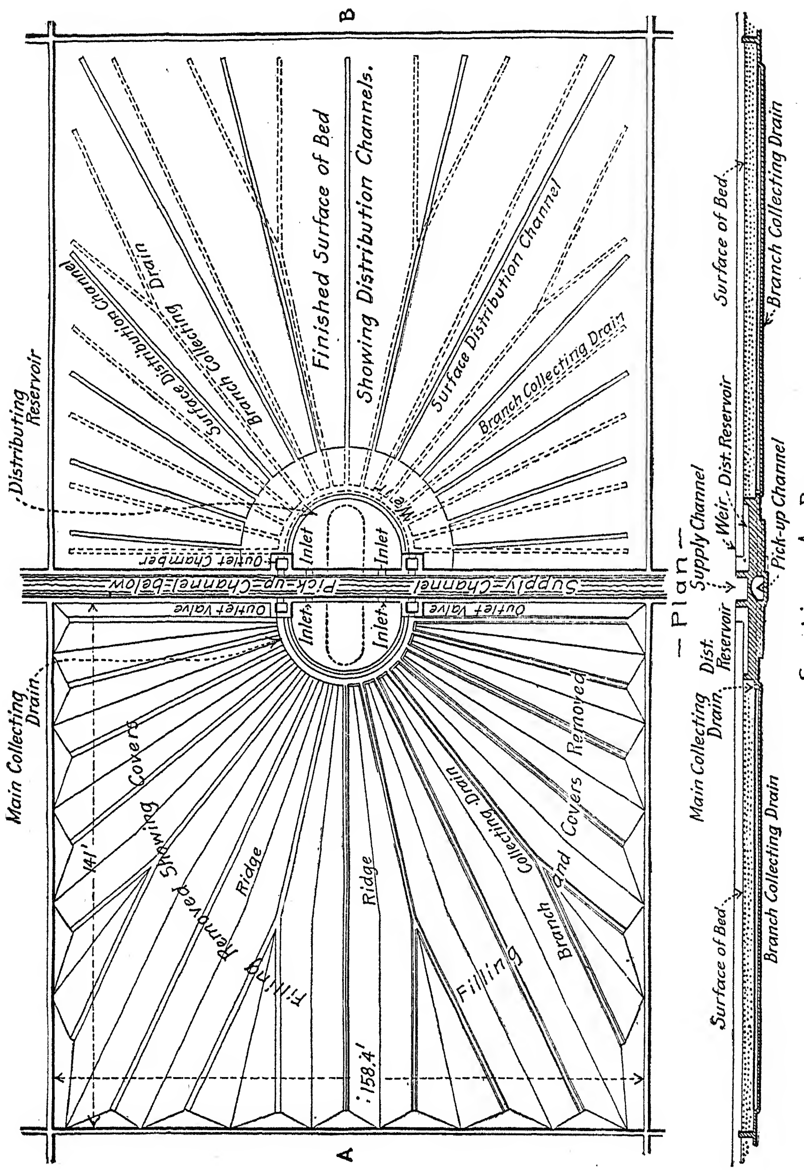


FIG. 191.—Plan of two half-acre contact beds, Manchester, England.

deep and covered to a depth of 4 to 9 in. with cinders from ½ to 1½ in. in size. Underdrainage systems similar to those used for trickling filters also have been employed.

Trickling Filters.—Trickling filters built in excavation without masonry floors are objectionable because the soil may become displaced by water flowing over it, thus causing settlement. Furthermore, the soil may become intermingled with the filtering material and impair the drainage of the filter. Such filters should be built upon floors of concrete or other masonry.

One of the earliest drainage systems consists of a concrete floor upon which is a layer of material composed of pieces preferably not less than 6 in. in diameter, the floor being sloped toward the main collectors. This coarse material affords better drainage than would the finer material of the bed, but clogging may result from the solids unloaded by the filter. The coarse material accomplishes little in the purification of the sewage because of the relatively small bacterial surface per unit of filter volume. In order that the solids may be carried away more readily with the effluent or flushed through the drains, and the gases of decomposition may have a ready means of escape, false floor systems are generally adopted.

The floor system used at Columbus and some other places (Fig. 192) is the least expensive type under some conditions. The concrete slopes toward the main drains, and the lateral drains of inverted half-round, slotted tile practically cover the entire floor. Floor slopes of at least 1 per cent should be secured. The tiles are bedded in cement to afford an even bearing, thus enabling them to carry the load of stone placed upon them. A layer of coarse filtering material about 4 to 6 in. in diameter is usually spread over the pipes to prevent the finer material from passing through the openings into the drains. Instead of slotted pipes of this kind half-round bell-and-spigot pipes laid with open joints have been employed. The opening should be made as large as possible without sacrificing the overlap of the bell. A 1-in. opening is desirable.

This floor system offers a fairly good opportunity for the passage of solids from the bed into the drains. If the bed is composed of coke, cinders, or other material likely to disintegrate, the valleys between the pipes may gradually become filled with fine material which may clog the drains. A disadvantage of this floor is that the flow of effluent is distributed over the floor

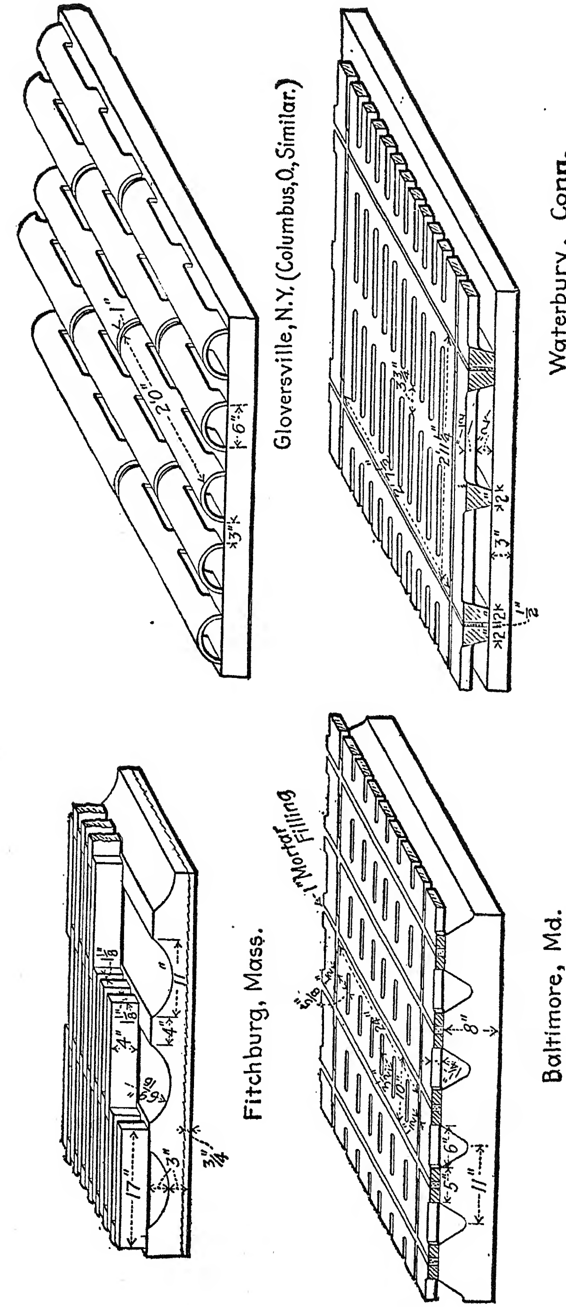


FIG. 192.—Types of floors for American trickling filters.

beneath the tile so that a relatively low velocity is afforded for carrying away the solids. Furthermore, it is not readily flushed because the water spreads over the bottom, rendering the stream ineffective.

Fig. 192 also shows the floor system designed for Waterbury, Conn. The concrete is flat, sloping toward the main drains. A special block of vitrified clay or reinforced concrete is set on the floor, furnishing a false bottom over the entire area. The drainage area is much greater than in the case of the Columbus type and the effluent flow is confined to the drains. The effluent is allowed to spread out over a large part of the floor, however, so that the self-cleansing qualities and facilities for flushing are not ideal.

The Baltimore floor system (Fig. 192) consists of a series of grooves and ridges in the concrete bottom. The grooves are covered with slotted vitrified clay or concrete slabs bedded in cement, concentrating the flow of effluent in narrow channels. The floor of the filter slopes toward the main drains and the inverts of the lateral drains are parallel with the floor, which necessitates a varying depth of stone in accordance with the slope of the floor. While the practical objections to such a variation in depth of filter may be questionable, it is theoretically an advantage to build the bed of uniform depth. With this type of floor system it is feasible, although possibly somewhat more expensive, to lay the floor level and give the necessary grade to the drains. In this case the filtering medium will be of uniform depth.

A type of floor designed by the authors for use at Fitchburg, Mass., and subsquently installed at other plants, is shown in Fig. 192. The lateral drains in the concrete are covered with narrow cement beams, affording a large percentage of drainage area as well as a concentrated flow of effluent. Cobble stones, which were abundant at the site, were placed by hand over the openings before placing the crushed stone on the bed.

It is evident that the thickness of the floor has an important bearing on the cost of the filter. The percentage of open area in the false floor for drainage and the proportion of free space for carrying away the effluent and gases of decomposition should be as large as can be provided at reasonable cost. The strength of the false floor should be ample for the load coming upon it.

The possibility that the drains may become clogged with solids unloaded by the filter or by organic growths in them necessitates ample provision for flushing. In some cases the upper ends of the laterals are carried through the filter wall, affording an opportunity for flushing, and in other cases flushing galleries have been provided. The size of the lateral drains should be sufficient to carry away the effluent promptly, to make flushing possible and to afford space for the circulation of air. Where tiles or channels are carried through the outer walls it is impossible to flood or submerge the filter for the purpose of destroying the larvae of the filter fly. This is also true where walls are not watertight or where they are omitted, the ballast taking its own angle of inclination.

The main drains may be rectangular or semicircular channels covered with slabs, or they may be circular. An open channel around the outside of the filter serving as a main drain, Fig. 186, affords an excellent means of ventilation. The grades of all parts of the drainage system should ensure freedom from backing-up of effluent in the ducts and channels.

383. Dosing Cycles.—Generally speaking, all processes of sewage filtration are intermittent in their operation, periods of applying the sewage to the filter alternating with periods of rest. The cycle of operation thus established is called the dosing cycle. In the case of intermittent sand filters the cycle may be measured almost in days, in that of contact beds in hours, and in that of trickling filters in minutes. The periods of rest incorporated in the dosing cycle must not be confused with those other periods of recuperation that may become necessary as a result of overloading of the beds or other untoward occurrences. The correct proportioning of dosing cycles is largely a matter of experience in the operation of filter units. The standards evolved are quite elastic and should be so considered.

Intermittent Sand Filters.—The rate of applying sewage to intermittent sand filters is commonly 1 cu. ft. per second for each 5,000 sq. ft. of area. Sewage is usually run on to a depth of about 3 in. over the surface. The quantity applied would be equivalent to a depth of somewhat over 3 in., the exact quantity depending upon the rapidity with which the bed would allow percolation to proceed. After dosing the sewage makes its way downward to the underdrains. Assuming a bed 1 acre in area, receiving on an average 75,000 gal. of sewage per day applied in doses equivalent to a depth of 3 in., there results:

Amount of sewage per filling....... $43{,}560 \times \frac{3}{12} \times 7.48 = 81{,}500$ gal.

Time elapsing between fillings, $\frac{81{,}500}{75{,}000} \times 24 = 26$ hours.

Rate of filling, $\frac{43{,}560}{5{,}000} = 8.7$ cu. ft. per second.

Time required for filling, $\frac{81{,}500}{7.48 \times 8.7 \times 60} = 21$ min

In this way the size and number of beds can be determined, taking into consideration the storage provisions as well as questions of bed maintenance and variations in sewage flow. The length of the actual resting period during which the bed stands empty will vary with the size of sand and extent of clogging. From 12 to 18 hours is a probable value. In some cases the rate of treatment and condition of the sand are such that the beds stand empty for 24 hours or more.

Contact Beds.—Contact beds are filled with sewage, allowed to stand full, emptied and allowed to rest. Then the cycle is repeated. Schedules vary according to the design and rate of sewage flow, the time of resting depending upon the number of fillings per day. The following schedule will illustrate such a cycle with two fillings per day:

	Hours
Time of filling	2.0
Time of contact	2.0
Time of emptying	2.0
Time of resting	6.0
Time of cycle	12.0

The cycle actually employed, however, will be determined by experience in operating under the conditions encountered at the particular plant. Basing computations upon the water capacity of the bed after it has been in operation for a considerable period of time, when the void space may have been reduced to about 20 per cent, the volume of sewage which can be treated by a bed 4 ft. deep and ½ acre in area will be $4 \times \frac{1}{2} \times 43,560 \times 7.48 \times 0.20 = 130{,}000$ gal. per cycle, or 520,000 gal. per acre per day. Allowing for a rest of 1 or 2 days in 5 or 6 weeks, the capacity is further reduced to about 500,000 gal. per acre per day.

It is difficult to determine the proper number of fillings per day in advance of actual experience with a plant, for this depends upon the quality and strength of the sewage, the quantity and

character of its suspended matter, the size and character of the ballast, the extent to which the ballast is clogged, and the character of the effluent desired. The stronger the sewage and the greater the quantity of suspended matter in it, the smaller will be the quantity which the contact bed can convert into an effluent of good quality. A fine contact material cannot treat as great a quantity as can material of medium or coarse size, although for a time at least the quality of the effluent produced may be decidedly better. Clogging the interstices of the ballast results in a loss of capacity, for it becomes necessary to operate the bed with small quantities of sewage and allow long periods of rest. Generally it is considered that the most important biological purification of the organic matter takes place while the bed is standing empty, and that this period should be made as long as possible, taking into account the necessary time for filling, standing full, and draining. British experience has indicated that 4 hours of rest are enough for average conditions, but this statement applies only to matured beds and does not take into account any periods of complete rest for a day or more when the ballast has become unusually clogged.

As filling and draining have no biological significance, it is generally held that the more quickly they take place the better, so long as the movement of the liquid is not sufficiently rapid to disturb the materials in the bed or the film adhering to them.

The period of standing full should not be so long that appreciable anaerobic decomposition takes place. In some cases Clark and Gage found[1] no marked difference when the period of contact varied from 0 to 5 hours, but, on the whole, periods exceeding 2 hours gave inferior effluents. Johnson found at Columbus that even 1 hour was too long a period to give the best results. The Royal Commission on Sewage Disposal stated in its Fifth Report that 2 hours contact generally gave the best results in the practical operation of contact beds, but added that no rule of general applicability could be laid down.

Trickling Filters.—Intermittency of operation, though frequently not essential, is secured in trickling filters (1) by the movement of the rotating or traveling dosing mechanism over the bed, (2) by intermittent discharge of sewage on to splash plates or through nozzles, varying the head on the nozzles during the dosing period, or (3) in the case of small units by the movement

[1] Report, Mass. State Board of Health, 1908, p. 445.

of the tipping-tray mechanism. In general the cycle is completed in 5 to 15 min., with approximately equal periods of rest and sewage application. Resting periods of too great length should be avoided to prevent drying or freezing of the filter and excessive loading at times of dosing, entailing less satisfactory dispersion of the sewage through the bed. There seems to be a general tendency to reduce the period of rest to a minimum when nozzles are employed which operate under varying heads, because the method of operation ensures sufficient intermittency for different parts of the bed.

There appears to be some difference of opinion as to the value of periods of recuperation of longer duration than the rest periods occasioned by the dosing cycle. The experience of the authors has been that a recuperation period of 24 hours or more is of great assistance at times in aiding trickling filters to unload accumulated solids. Similar resting in rotation of portions of filters affected by organic growths has sometimes proved of marked benefit. The length of the recuperation period will be governed by the extent of the clogging and the results obtained by actual operating tests.

The hydraulics of nozzles and dosing tanks in relation to dosing cycles will be considered in Sections 385 and 386.

384. Automatic Dosing Apparatus.—The dosing cycles of sand filters and contact beds are controlled readily by hand. The dosing of trickling filters, however, usually is made automatic. In considering the advisability of automatic control for sand filters and contact beds, it must be remembered that both the quantity and quality of sewage vary greatly from hour to hour and from day to day, and that the capacity and efficiency of the beds change from time to time. In a small plant, with from three to five beds, it is possible that the same bed may receive day after day the strong sewage of the day time, while the other beds, receiving the weaker sewage, are called upon to do less work. In most cases, however, the fluctuations in flow from day to day will cause more or less change in the hourly cycle, and under such conditions the use of automatic apparatus for dosing will probably insure better work on the part of the beds and less expense for caretakers than the operation of the controlling gates by manual labor. In large plants the complication of the apparatus is so great and the beds vary so much in their capabilities that it is probable that better results can be

obtained by the intelligent operation of gates by manual labor than by automatic dosing apparatus.

Generally speaking there are three classes of apparatus for automatic dosing of sewage filters: (1) air-controlled siphonic apparatus, (2) mechanically controlled siphonic apparatus and (3) mechanical devices. For use with intermittent sand filters and trickling filters these devices are installed only for the purpose of applying sewage to the beds. In contact beds they are further used for emptying the beds after a definite period of contact has elapsed.

Air-controlled Siphonic Apparatus.—The main feature is the siphon. Its principal parts (Fig. 193) are the main trap, a pipe

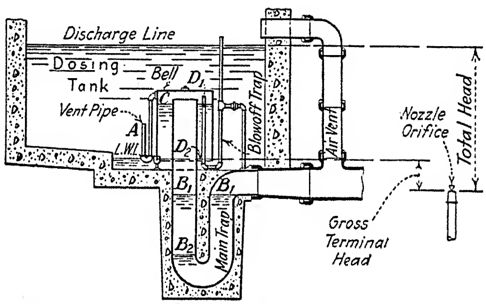


FIG. 193.—Sewage siphon with adjustable blowoff trap, deep seal type (*Miller*).

casting with the long leg extending above the bottom of the dosing tank and the short leg connected to the discharge pipe to the filter; the bell, a cylindrical casting set over the long leg of the main trap and supported on legs or piers above the tank floor; the vent pipe, and the blowoff trap, made up of small galvanized wrought-iron pipe.

The main trap, immediately after the siphon has ceased discharging, stands full of water to the level B_1. The blowoff trap is also full to the level D_1. The vent pipe is empty. Sewage flows into the dosing tank and the water level rises until the open end of the vent pipe is reached at A. The vent pipe then becomes full of water and the siphon is sealed against the escape of air confined in the bell and upper part of the long leg of the main trap. As the water in the dosing tank continues to rise, it exerts a pressure upon the air confined in the siphon and forces the water in the long leg of the main trap down toward the lower bend. The water in that portion of the blowoff trap under the bell is likewise forced down. At the same time the water level inside the bell rises. Just before the discharge level in the dosing tank is reached, the water level in the blowoff trap is at D_2, in the

main trap at B_2, and in the bell at C. A slight further rise of the water in the dosing tank forces the seal in the blowoff trap, thus releasing the air confined in the bell and causing a sudden inrush of water from the dosing tank into the bell, which sets the siphon into full operation. The sewage in the dosing tank is discharged through the siphon until the level is at the low-water line at the lower bend of the vent pipe, when air is drawn into the bell through the vent pipe, the siphonic action is broken, the bell is filled with air, the discharge ceases, and the main trap and blowoff trap are refilled with water. The dosing tank then fills again and the siphon is ready for another discharge.

The air vent in the discharge pipe line, although not necessary for the working of the siphon, allows the escape of air previously confined in the bell and prevents trouble from air in the pipe line. The small siphons, 3 to 8 in. in diameter, in some cases do not require blowoff traps to ensure their working.

Where it is desirable to dose two or more filter beds in rotation, this can be done by installing several siphons, each connected to a filter bed and arranged to discharge in rotation automatically. Two siphons of the type illustrated by Fig. 193, set side by side in a dosing tank, will operate alternately without special piping. For three or more alternating siphons, a special system of piping with starting bells or other controlling devices is required.

For use with trickling filters a trapless siphon is now also on the market, the distribution system being kept at such an elevation as to act as the seal. The head lost in this type of siphon is smaller, especially as larger siphon sizes are approached.

The discharge of siphons can be represented by the common orifice formula $Q = C_s a_s \sqrt{2gh_s}$. Here h_s represents the head causing flow, a_s the area of the siphon and Q the rate of discharge. For free discharge this head is measured from the surface of the sewage in the tank to the center of the outlet from the main trap (Fig. 193). When the discharge is not free, when sewage is discharged into the distribution system of a trickling filter for example, h_s is the head lost in the siphon. The coefficient of discharge C_s is approximately 0.6. In emptying sewage from a tank, however, there is a delay in bringing the siphon into full operation and in overcoming the inertia of the sewage in the distribution system. During this time there is a fall of sewage level in the tank. This effect produces what is called the "dosing tank loss" which, according to investigations of the Pacific Flush Tank Co., is measured approximately by the relation

$$h_d = 364\frac{a_s^2}{AQ}$$

where h_d = dosing tank loss in feet
Q = rate of discharge in cu. ft. per second
A = area of dosing tank at maximum level in sq. ft.
a_s = area of siphon in sq. ft.

To determine the characteristics of the dosing device, therefore, both these relations must be taken into account (Section 387). Where a single tank is used allowance also must be made for the volume of sewage flowing into the tank while the siphon is discharging.

Air-lock Feeds.—Fig. 194 shows in outline the general features of an assembly of siphons and air-lock feeds for four contact beds. At the beginning of a cycle of operation the wells in front of the air-lock feeds are filled with water, except the one that is to operate first. The blowoff traps are all filled with water.

Sewage entering the channel from the settling tanks flows through the feed (the one not sealed with water) into the bed until it is filled, or until the sewage level reaches the top of the withdraw siphon. A slight additional rise in water level causes the withdraw siphon to come into operation, and the compression chamber is filled through the withdraw siphon until the sewage level inside is the same as in the bed. As the compression chamber fills, the air in the compression dome is put under pressure and forced into the upper part of the feed, gradually displacing the sewage flowing through the feed until the sewage level is forced down below the inside crest of the feed, when the flow through the feed ceases and the feed is air-locked.

The same rise of sewage in the compression chamber also produces a pressure in the starting bell, which is transmitted to the blowoff trap of the feed next to operate. Just before feed (1) is air-locked the seal in blowoff trap (2) is forced, thus releasing the air confined in feed (2), and allowing the sewage to discharge into contact bed (2). This prevents any backing up in the distributing channel or settling tanks.

After standing for the required time, the sewage in the bed is discharged by a timed siphon, described later. As the sewage level in the bed falls, siphonic action is started in the withdraw siphon and the compression chamber is drained. By this means the compression dome and starting bell are vented, the blowoff trap is filled and the chamber is ready for the next cycle of operation.

Hydraulically, feeds may be considered as siphons and the discharge through them may then be expressed by the orifice formula, $Q = C_f a_f \sqrt{2gh_f}$. Assuming a coefficient $C_f = 0.7$, the head required to operate a 6-in. air-lock feed one foot in length is given by the following relationship,[1] $h_f = 0.127Q^2$, where h_f is expressed in feet and Q in cu. ft. per second.

[1] The Pacific Flush Tank Co. in its Catalog 30 gives losses of head for air-lock feeds which may be represented by the equation, $h_f = 0.129Q^{1.64}$.

Timed Siphons.—To render the operation of contact beds completely automatic they require apparatus controlling the

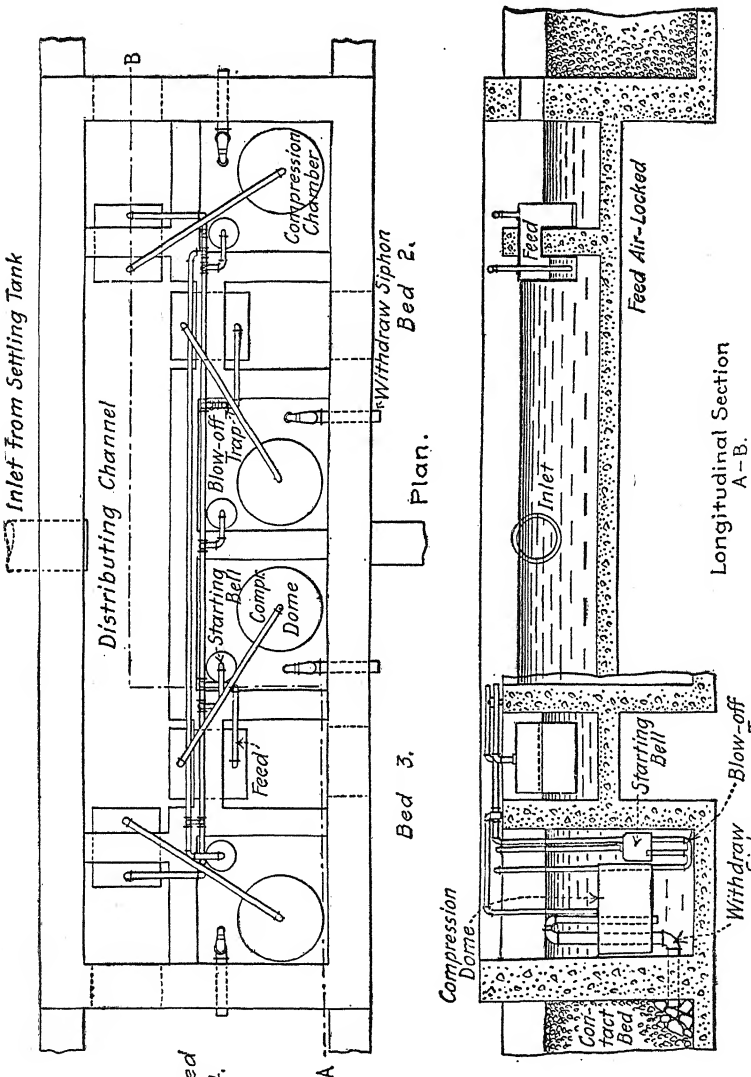


Fig. 194.—Air-lock feeds for contact beds (Miller-Adams).

time the sewage stands in the beds. Timed siphons are frequently used for this purpose. The general details of the apparatus are shown in Fig. 195.

At the start, the main trap, the blowoff trap and tile well in the timing chamber are filled with water. The size of the timing chamber and the size of the opening in the timing valve determine the period of contact and require trial and adjustment to obtain the specified period of contact. The timed siphon is controlled by the starting bell in the timing chamber, and until there is sufficient pressure in the starting bell to force the seal of the blowoff trap, the siphon will not discharge and the bed will stand full. The timing valve is located below the full water level in the contact bed, so that when the bed is full there is a continuous discharge through the timing valve into

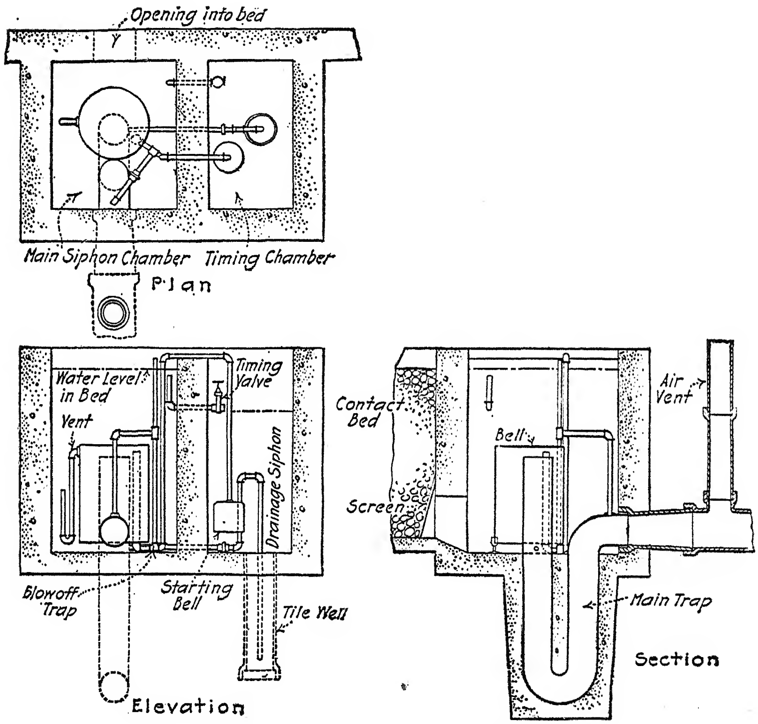


FIG. 195.—A timed siphon (Miller).

the timing chamber. The siphon receives air through the vent when the sewage has been drawn down to the low level and the discharge then ceases. When the timed siphon is operating, the draining siphon is discharging the sewage in the timing chamber, and at the end of the discharge the starting bell is vented, the timing chamber emptied and the apparatus is ready for the next filling of the bed.

The total head required to operate contact beds equipped with air-lock feed and timed siphon ranges from about 12 in. for the smallest installations to 21 in. for the largest in addition to the depth of the bed.

Mechanically-controlled Siphonic Apparatus.—Several kinds of mechanical devices are used for directing the discharge of siphons to one pipe after another. Figs. 196 and 197 illustrate one used at a number of sewage treatment works by Barbour.

The main features are an air-lock siphon controlled by an air valve and float, and a revolving cylindrical gate valve, also actuated by a float, the whole apparatus being set in the dosing tank. As the level of the sewage in the dosing tank rises, it lifts the large float A to which is attached a rack and pinion, causing the cylindrical gate valve to turn until the opening in the

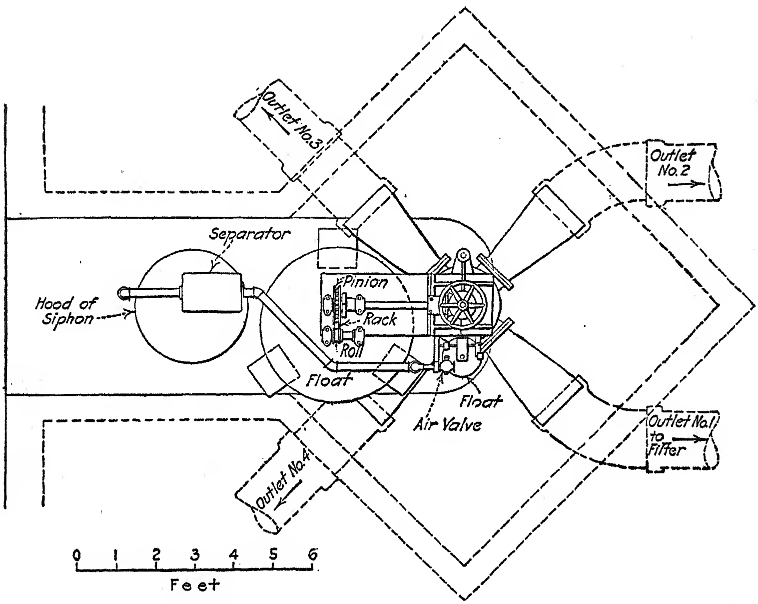


FIG. 196.—Plan of Barbour dosing apparatus at North Attleborough, Mass.

gate is opposite one of the outlet pipes to a filter bed or group of filter beds. At the same time, float B rises until the high-water level in the tank is reached, when the trip E on the float rod opens the air valve D, suddenly reducing the air pressure in the siphon bell and causing the siphon to come into full discharge. In case the trip failed to open the air valve, an additional rise in the water level would bring the siphon into full discharge by the aid of the pilot pipe, in a manner similar to that previously described. The siphon continues to discharge until the low-water level in the tank is reached, when air is drawn in under the edge of the siphon bell, the siphonic action is broken, and the discharge ceases. When float A falls with the sewage level, the cylindrical gate valve is not turned back, but is held in place by a pawl and ratchet on the pinion. The tank then fills again, raising float A, which turns the cylindrical gate valve around to the next outlet

pipe, and the process is repeated as described. The advantage of the small float B and air valve is that the size of the dose can be regulated by setting the float B higher or lower on the float rod. The action is positive and productive of more satisfactory results than when the pilot pipe is relied upon to start the siphon. The "separator" is necessary to prevent suspended matter from getting into the air valve.

Mechanical Dosing Apparatus.—An example of mechanical dosing apparatus is shown in Fig. 198. Sewage flows through the settling tank into a dosing chamber containing a float. A chain

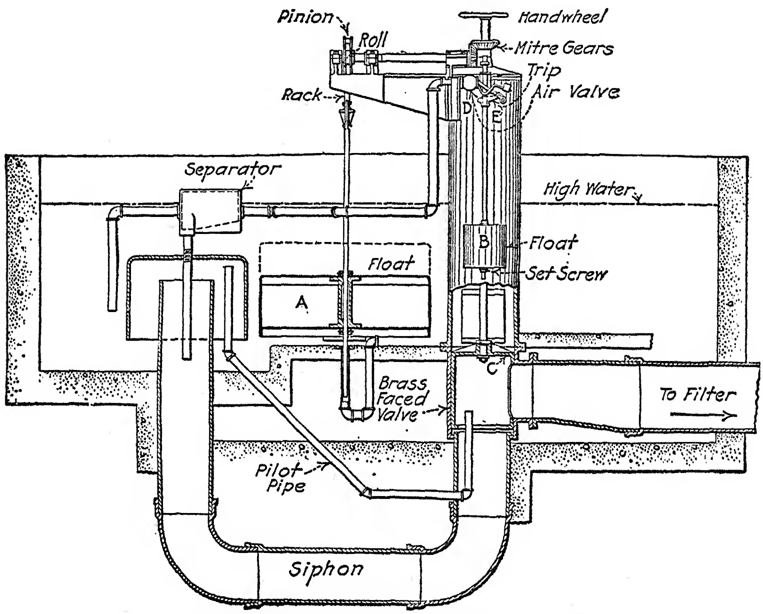


FIG. 197.—Sectional elevation of Barbour dosing apparatus at North Attleborough, Mass.

attached to the float passes over a sprocket wheel, on the shaft of which is a weighted lever. As the sewage in the dosing tank rises, the float rises and the shaft revolves, bringing the weighted lever to a vertical position. As soon as the shaft turns so as to bring the center of gravity past the vertical, the weighted lever falls on the opposite side, and the flap valve between the dosing chamber and the distributing chamber is suddenly opened. Each rise of the float revolves the countershaft, to which the distributing gates are attached, a fifth of a revolution, the fall of the float failing to revolve the shaft in the opposite direction

on account of the pawl and ratchet. The five flap valves leading to the five filter beds are each attached to the shaft at points equally spaced around the circumference, and thus at each one-fifth turn of the shaft a new gate is opened, the other four remaining closed. In this way the doses are distributed to each filter bed in succession.

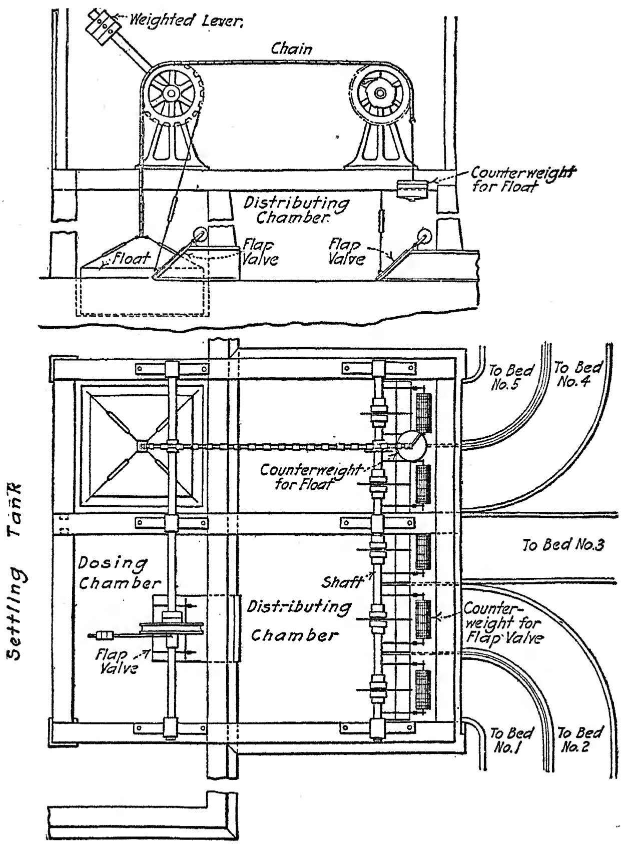


FIG. 198.—Dosing apparatus at Newton, N. J. (Ansonia).

This apparatus is readily adjustable and in general has been found to work satisfactorily. Care should be exercised in designing apparatus of this type to provide parts of sufficient strength to withstand the shock due to the suddenly applied force of the falling weight and the resulting jerk on the flap valves.

385. Hydraulics of Nozzles for Trickling Filters.—The hydraulic characteristics of various types of nozzles have been studied

by the manufacturers, such as the Pacific Flush Tank Company,[1] by the Experiment Station of Purdue University,[2] by the authors[3] and by others. Fig. 199 shows the distribution curves obtained in the Purdue experiments for a Worcester-type nozzle. Generally speaking, the discharge through spray nozzles can be represented by the standard orifice formula, $Q = C_n a_n \sqrt{2gh_n}$. The coefficients (C_n) obtained at Purdue for the Worcester nozzle range from 0.66 at heads of 8 ft. to 0.69 for

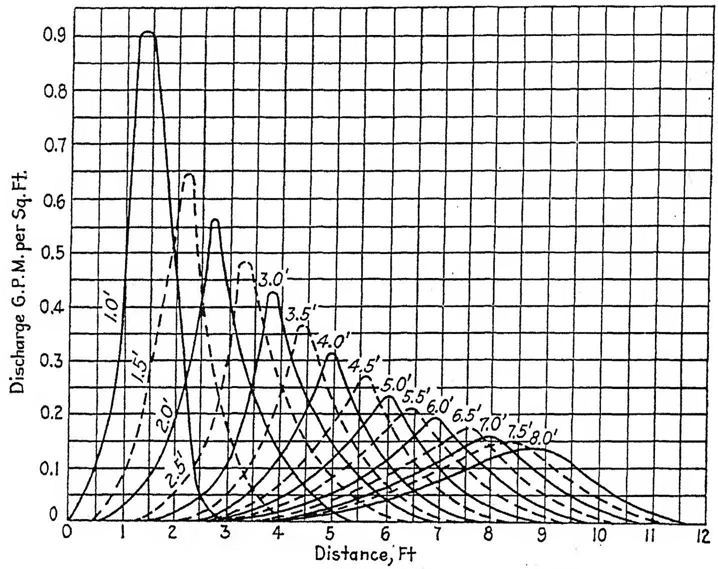


FIG. 199.—Distribution curves for 0.813-in. Worcester circular nozzle under different constant heads.

heads of 1 ft. The discharge of Worcester-type nozzles is shown in Fig. 200.

Circular-spray nozzles which are not spaced to give overlapping sprays cover only 78½ per cent of a rectangular area when placed at the centers of contiguous square areas, and only 90.1 per cent of the bed when arranged at the apices of equilateral triangles. Such a waste of filter area is undesirable. Accordingly the usual arrangement is to place the nozzles at the apices of equilateral triangles and provide for some overlapping of the

[1] Catalog 30.
[2] Bull. 3 and 20.
[3] "American Sewerage Practice," Vol. III.

sprays, as shown in Fig. 201. When fixed nozzles are used and the head remains constant, the quantity of liquid falling upon a

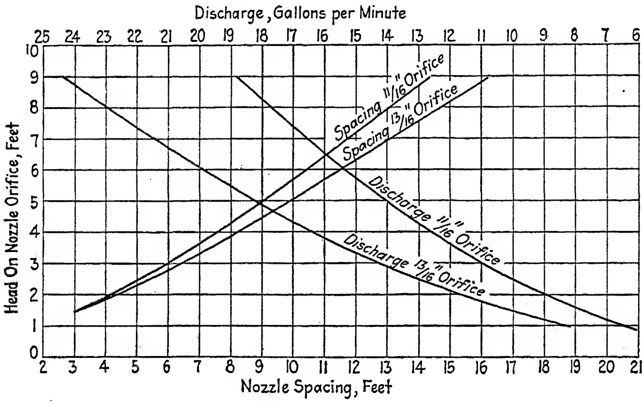


FIG. 200.—Discharge and spacing of Worcester-type nozzles.

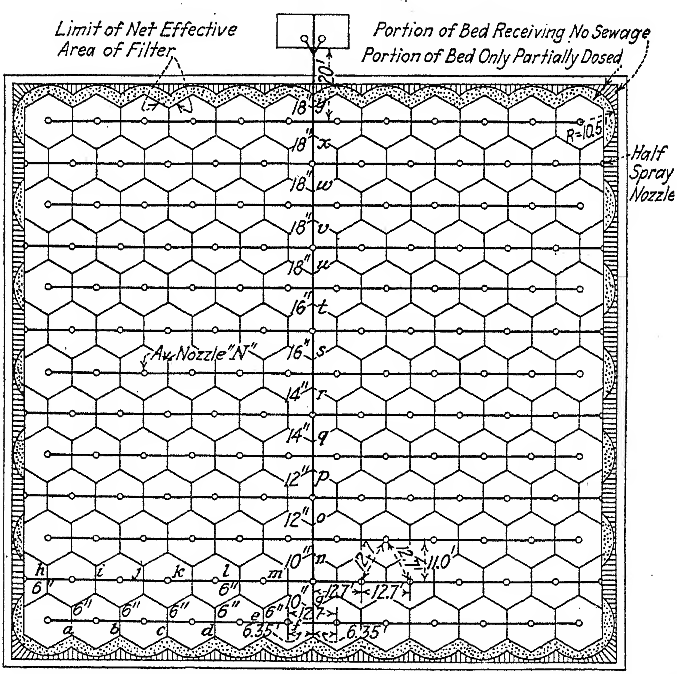


FIG. 201.—Arrangement of nozzles and distribution system for a trickling filter.

unit area at the periphery of the wetted area is less than that at some lesser distance; and still nearer the nozzle the rate of fall is

again less than the maximum. Uniform distribution over the surface therefore necessitates overlapping of the sprays at the time of maximum head, as well as a variation in head upon the nozzles.

While square-spray nozzles apparently will overcome the necessity for overlapping sprays, wind action and nozzle clogging affect all sprays in such a manner as in a measure to offset refinements of design of nozzles. Circular-spray nozzles seem to be preferred by operators generally.

Nozzle spacing has been worked out for the most common types of nozzles from studies such as those illustrated in Fig. 199 and from tests of experimental and existing installations. Allowing for overlapping of sprays and placing the nozzles at the apices of equilateral triangles, the net effective area of each nozzle is hexagonal in shape (Fig. 201). Along the edges of rectangular beds there remain areas (stippled in Fig. 201) which are but partly dosed, and other areas (cross-hatched in Fig. 201) that receive no sewage whatever. Use of "half-spray" nozzles along the edges has been resorted to (Fig. 201). These are generally made by covering one-half of the nozzle opening and attaching flanges which will direct the spray inward.

Assuming that the capacity of a trickling filter is approximately proportional to its depth, a smaller volume of filtering material may be provided around the edge of a filter, or, in other words, the bottom of the filter may be made smaller in area than the top, thus effecting a considerable saving in filtering material. For example, at Fitchburg the length and width of the filters at the bottom are 5 ft. less than at the top.

Complete utilization of the marginal strip is not practicable if wind-blown spray is to be kept on the filter. Marginal allowances of 18 to 24 in. are frequently provided. Selection of nozzle spacing is illustrated by an example in Section 387.

386. Hydraulics of Dosing Devices for Trickling Filters.—In order to obtain the variation in head on the nozzles of trickling filters which is necessary to distribute the sewage as uniformly as possible over the bed, use is made of (1) dosing tanks or (2) head-varying valves.

Dosing Tanks.—The usual form of dosing tank is hopper shaped or is tapered by the use of sloping side walls or steps, so that the capacity of the upper portion of the tank is much greater than that of the lower portion. This causes the nozzles to

discharge longer under the higher heads and throw a larger proportion of the spray on to the more extensive outer areas. The operating head commonly ranges from a maximum between 5 and 10 ft. down to a minimum of 1 or 2 ft. Automatic air-lock siphons are used in conjunction with these tanks and are similar to those employed in dosing intermittent sand filters.

In the design of dosing tanks the following three types must be distinguished:

1. Single dosing tanks into which sewage flows continuously, the siphon capacity being such that the rate of outflow when discharging always exceeds that of inflow, even under the minimum operating heads and with maximum inflow. If properly proportioned these tanks will alternately fill and discharge.

2. Twin dosing tanks with air-lock feeds discharging to a common nozzle field (Fig. 202); one tank fills while the other discharges and stands empty during a brief rest period. Deep bells on the siphons or flap valves shut off intercommunication between the tanks. The maximum rate of inflow must be such as to fill the tanks in a longer time than is required for discharge.

3. Twin dosing tanks with air-lock feeds discharging on to separate nozzle fields, each half of the field having its separate dosing cycle. Since only one-half the number of nozzles are supplied at one time, the dosing tank size is thus cut in two.

Twin dosing tanks assure the same size of dose and the same distribution of the sewage on the filter at each dose, regardless of the rate of sewage flow, until a maximum storm-water rate is reached at which one of the tanks will go into continuous operation. By using two dosing tanks which operate alternately, the point at which continuous operation begins is kept higher. Such an installation at Fitchburg, Mass., is illustrated in Fig. 203.

The design of a dosing tank and nozzle field is exemplified in Section 387. Suitable design must take the following hydraulic factors into account as relating to flow and losses of head:

1. Friction in air-lock feeds and over inlet weirs.
2. Siphon losses.
3. Dosing tank losses.
4. Velocity heads and friction losses in distributing pipes.
5. Entrance losses, losses in passing openings, and losses at change in section and direction.
6. Losses in gates.

Losses 2 to 6 commonly equal from 15 to 35 per cent of the maximum available head on the nozzles, with 25 per cent as a

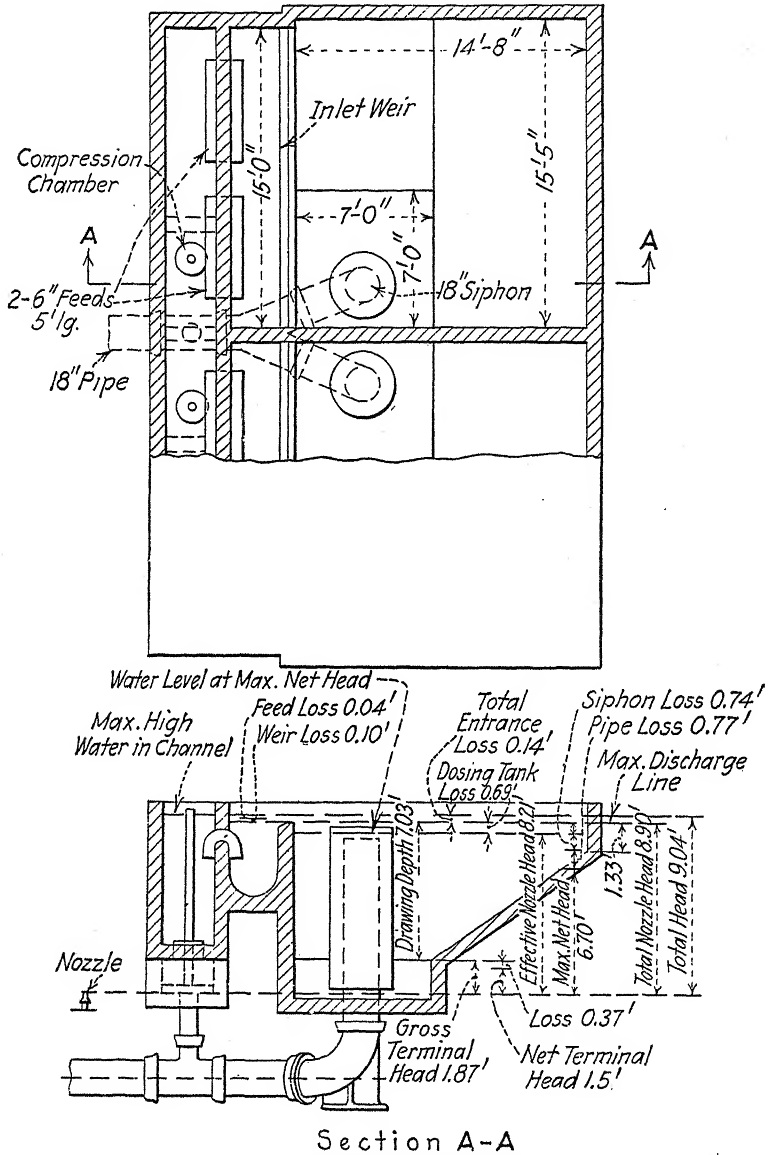


FIG. 202.—Twin dosing tanks with air-lock feeds and deep-bell, trapless siphons.

fair average. These losses are divided about equally between the siphon, dosing tank and distribution system. Losses in the latter can be reduced greatly by using rounded orifices, bell mouth openings and gradual changes in section.

The Pacific Flush Tank Co. has studied the operation of trickling-filter dosing-systems using the company's dosing apparatus and has derived formulas for the computation of the special losses involved; these the authors have reduced to general terms as given below. The formulas were derived from observations at five trickling filter plants using siphons from 12 to

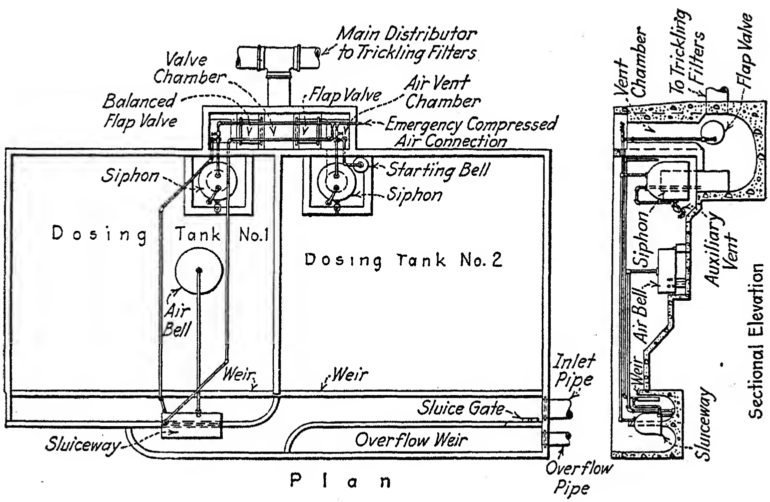


FIG. 203.—Dosing tanks and siphonic apparatus, Fitchburg, Mass.

30 in. in diameter, operating under a "total head" of 7 to 10 ft. and using dosing tanks with maximum area of water surface from 200 to 800 sq. ft. The formulas naturally apply only to the company's apparatus. The expression for siphon loss (h_s) holds, furthermore, only for siphons of the trapless type, and the formulas for dosing-tank loss (h_d) and time of discharge (t) should be used only when the maximum velocity in the siphon is between 2 and 4 ft. per second. It will be noted that the expressions are empirical, with the exception of that for siphon loss. So far as the authors know, these are the best data to be had on the subject.

$$h_f = 0.129\left(\frac{Q_i}{L_f}\right)^{1.64} \tag{1}$$

$$h_w = 0.470\left(\frac{Q_i}{L_w}\right)^{0.565} \tag{2}$$

$$h_s = \frac{1}{2g}\left(\frac{Q}{0.6a_s}\right)^2 \tag{3}$$

$$h_d = \frac{364a_s^2}{AQ_{\text{max.}}} \tag{4}$$

$$t_d = 454\left(\frac{a_s}{Q_{\text{max.}}}\right)^2 \tag{5}$$

Nomenclature:

h_f = loss of head in 6 in. inlet feed, ft. (inlet feed loss).
h_w = loss of head over inlet weir, ft. (Weir loss).
h_s = loss of head in dosing siphons, ft. (siphon loss).
h_d = loss of head starting, ft. (starting or dosing tank loss).
t_d = time elapsed from time siphon starts to time of maximum discharge (time during which dosing tank loss occurs), sec. (starting time).
Q_i = rate of inflow to tank, cu. ft. per second.
Q = rate of discharge, cu. ft. per second.
$Q_{\text{max.}}$ = maximum rate of discharge, cu. ft. per second.
a_s = cross-sectional area of siphon, sq. ft.
A = area of dosing tank at maximum water level, sq. ft.
L_f = length of inlet feed, ft.
L_w = length of inlet weir, ft.

Generally good distribution will be secured if the water level in the dosing tank falls at a uniform rate, or nearly so. The surface area of the tank, though determined to some extent by considerations of nozzle distribution, is even more influenced by the dosing tank loss. Since this loss is volumetric and must be limited to a certain value it generally controls the choice of the tank area (see Section 387). This maximum area must be carried down a distance equal to the dosing tank loss. From here on the area must decrease in proportion to the rate of discharge. A parabolic curve will result and is generally approximated by continuing the vertical sides of the tank for a short distance and then sloping the sides in a straight line to the low water line.

Head-varying Valves.—In 1906 Stearns suggested using a mechanically operated butterfly valve in the delivery pipe to trickling filters as a means of varying the head on the nozzles, and about the same time Weand designed float-operated butterfly valves for use with the trickling filters at Reading, Pa.

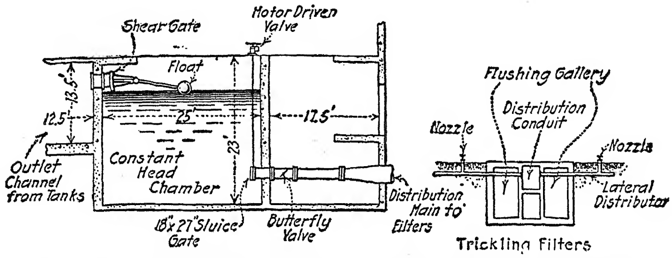


FIG. 204.—Sectional elevation of dosing apparatus, Baltimore.

One of the largest filter plants employing this means of regulating head is that at Baltimore, where the head on the nozzles runs from almost zero to 9 or 10 ft. The installation is shown diagrammatically in Figs. 204 and 205. There are ten 18- by 27-in. rectangular conduit castings, one to each filter bed, leading from the constant head chamber to the distribution system of the beds.

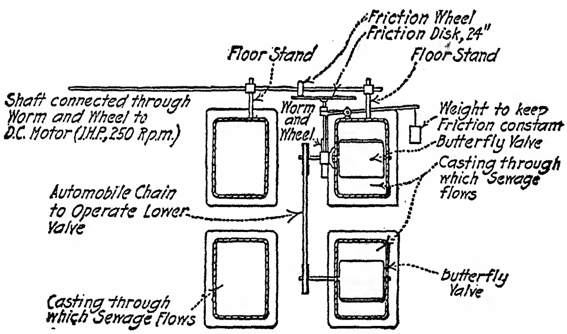


FIG. 205.—Operating device for butterfly valve, Baltimore.

The opening to each of these castings is controlled by an electrically operated sluice gate. Each motor is connected to an autostarter actuated by a trip on a float rod extending upward from a float in the outlet channel. The trips on these rods are set at four different levels so as to bring sections of the trickling filters

into operation one after the other by opening the motor-operated valves. For example, if two units are in operation and sewage begins to back up in the outlet channel faster than it can be taken care of by the area of trickling filters then in service, the float under the third auto-starter is raised sufficiently to trip the latter and open a sluice gate on the inlet line to section 3 of the trickling filters.

Inside each rectangular casting above referred to is a flat iron plate swung on its horizontal axis by a shaft extending through a stuffing box to the outside of the casting. This plate fits the inside dimensions of the casting with a clearance of about ½ in. The end of the shaft is geared to a 1-hp. direct-current motor. The plate is turned through 1 revolution (2 cycles) approximately every 7 min., thus alternately damming and releasing the sewage flow to the nozzles. The spray on the filters under this action falls first on the stone immediately adjacent to the nozzle, gradually increasing its breadth to a distance of 7½ ft. from the nozzle, at which moment the flat plate in the casting has turned till it is in a horizontal position and the maximum volume of sewage is flowing.

387. Outline of Hydraulic Computations for a Trickling Filter.—The following outline is suggestive of one of a number of approaches that may be taken in the solution of the hydraulic problems connected with the design of the dosing devices and distribution system of a nozzle field. An hypothetical problem is set, rather than an actual case, because it can be presented more simply and will illustrate the issues more readily. Repeated trials of different layouts and studies of individual parts of the system, involved in the economical design of filters in practice, are not given, in order to conserve space.

Assumed Basic Data.

Average volume of sewage to be treated.......	1.0 m.g.d.
Maximum rate of flow.......................	3.0 m.g.d.
Minimum rate of flow.......................	0.5 m.g.d.
Loading of filter...........................	0.333 m.g.d. per acre-foot
Depth of filter.............................	6 ft.
Difference in elevation between maximum discharge line of dosing tank and nozzle openings	8.9 ft.

Computations for Nozzle Field.—Let it be assumed that the nozzle field is to be constructed with cast-iron pipes laid near the surface of the bed, and that circular-spray Worcester-type nozzles are to be employed (Fig. 200).

Selection of Nozzles.

1. The economical range in the maximum net head on the nozzles of a trickling filter is commonly found to extend from 65 to 85 per cent of the total head. For a head of 8.9 ft., therefore, it will generally lie between 5.8 and 7.6 ft. *Try* 6.7 *ft.*

2. The minimum net head or net terminal head is ordinarily kept between 1.0 and 2.5 ft. *Try* 1.5 *ft.*

3. Circular-spray Worcester-type nozzles will have the following characteristics when operating under heads of 6.7 and 1.5 ft.:

Nozzle characteristics	Size of nozzle, in.		Remarks
	$1\frac{1}{16}$	$1\frac{3}{16}$	
1. Nozzle spacing, ft.	11.4	12.7	From Fig. 200
2. Spacing of laterals, ft.	9.9	11.0	0.866 × nozzle spacing
3. Number of nozzles required	193	156	$\frac{1.0 \times 43{,}560}{0.333 \times 6 \times (1) \times (2)}$
4. Maximum rate of discharge, per nozzle, g.p.m.	16.2	21.0	From Fig. 200, or using
5. Minimum rate of discharge, per nozzle, g.p.m.	7.7	10.0	$C = 0.66$ to 0.68 in $Q = Ca\sqrt{2gh}$
6. Maximum rate of dosing nozzle field, g.p.m.	3,130	3,280	(3) × (4)
c.f.s.	6.96	7.30	
7. Minimum rate of dosing nozzle field, g.p.m.	1,490	1,560	(3) × (5)
c.f.s.	3.31	3.48	
8. Average of maximum and minimum dosing rates, c.f.s.	5.14	5.39	½[(6) + (7)]
9. Maximum rate of sewage flow, c.f.s.	4.64	4.64	3.0 × 1.547

Try using a $1\frac{3}{16}$*-in. nozzle.* The average of the maximum and minimum dosing rates of this nozzle is well in excess of the maximum rate of sewage flow. Its spray limit is 10.5 ft. (from Fig. 199).

Selection of Distribution System.

4. *Try a distribution system layout consisting of* 13 *laterals, each with* $\frac{156}{13} =$ 12 *nozzles* (Fig. 201). A number of systems should be studied and the most economical one selected.

5. Computations for the head lost in the pipe system shown in Fig. 201 at the maximum rate of dosing are illustrated in the following schedule:

Pipe	Length, ft.	Diameter, in.	Sewage flow, c.f.s.	Velocity, ft. per second	Friction losses: Pipe losses[1] Ft. per 1,000	Pipe losses[1] Ft.	Passing openings,[2] ft.	Bends,[3] ft.
a	12.7	6	0.047	0.24	0.095	0.001		
b	12.7	6	0.094	0.48	0.34	0.004		
c	12.7	6	0.140	0.71	0.71	0.009		
d	12.7	6	0.187	0.95	1.22	0.015		
e	12.7	6	0.234	1.19	1.85	0.024	0.001	
f	6.35	6	0.281	1.43	2.60	0.017	0.001	T = 0.040 NOTE.—This is the loss to each lateral.
g	11.0	10	0.562	1.03	0.78	0.009		
n	11.0	10	1.12	2.06	2.82	0.031	0.004	
o	11.0	12	1.68	2.14	2.45	0.027	0.004	
p	11.0	12	2.25	2.87	4.20	0.046	0.008	
q	11.0	14	2.81	2.62	2.98	0.033	0.006	
r	11.0	14	3.37	3.15	4.18	0.047	0.009	
s	11.0	16	3.93	2.82	2.91	0.032	0.007	
t	11.0	16	4.49	3.22	3.72	0.041	0.010	
u	11.0	18	5.05	2.86	2.61	0.029	0.008	
v	11.0	18	5.62	3.18	3.18	0.035	0.009	
w	11.0	18	6.18	3.50	3.79	0.042	0.012	
x	11.0	18	6.74	3.81	4.42	0.049	0.013	
y	20.0	18	7.30	4.13	5.08	0.102	0.016	Y = 0.265

[1] By Hazen and Williams' formula, using $C = 100$.

[2] 0.03 × velocity head for each opening or branch. FLINN, WESTON and BOGERT; "Water Works Handbook," 1st. Ed., p. 583.

[3] *Jour.* N. E. W. W. A., 1913; **27**, 520. T loss = 1.25 velocity heads; Y loss = 1 velocity head.

6. The pipe loss (h_p) to the average nozzle "N" (Fig. 201) is the sum of the losses in lines c to f and s to y, together with the entrance T loss, the Y loss, and the loss in passing openings. It is found to be 0.77 ft. This is about ⅓ of the total loss assumed to be available (8.9 − 6.7 = 2.2 = 3 × 0.73) and represents a common value. Nozzles other than "N" should also be studied.

Computations for Dosing Tank.—Investigate the use of twin dosing tanks supplying a common nozzle field and equipped with air-lock feeds and deep-bell, trapless siphons (Fig. 202). Try a dosing cycle of 4.5 min. for the maximum rate of flow of 3.0 m.g.d. = 4.64 c.f.s. This is a common cycle time.

Size and Shape of Dosing Tank.

7. For a dosing cycle of 4.5 min. the effective volume of dosing tank is 4.64 × 4.5 × 60 = 1,250 cu. ft.

8. By following common practice and selecting a siphon of such size that the maximum siphon loss (h_s) is about equal to the pipe loss (h_p), about 1⁄12 of the total head or ⅓ of the total loss assumed to be available, it is found,

using a coefficient of discharge of $C_s = 0.6$, that the nearest size is 18 in. (Section 386). The siphon loss is, therefore,

$$h_s = \left(\frac{Q}{C_s a_s \sqrt{2g}}\right)^2 = \left(\frac{7.30}{0.6 \times 1.767 \times 8.02}\right)^2 = 0.74 \text{ ft.},$$

where Q is the maximum rate of dosing the nozzle field.

9. The effective nozzle head for maximum discharge is, therefore, 6.7 + 0.77 + 0.74 = 8.21 ft.

10. Assuming that the heads for other rates of discharge vary approximately as the squares of the rates, the minimum discharge head or gross terminal head will be $8.21 \times \left(\frac{3.48}{7.30}\right)^2 = 1.87$ ft. (In actual design this value would be checked by computations similar to those given for maximum discharge.)

11. The bottom width of the tank must be such as to accommodate the 18-in. siphon and to result in an economically proportioned tank. The siphon needs 6.67 ft. (from Pacific Flush Tank Catalog 30). *Try 7 ft.*

12. If we proportion the tank areas to the rates of discharge the top width becomes about $7 \times \frac{7.30}{3.48} = 14.67$ ft., because the tank length (L) is commonly held constant.

13. The dosing tank loss (h_d) may now be expressed in terms of the top area or length by applying the formula suggested by the Pacific Flush Tank Co.:

$$h_d = 364\frac{a_s^2}{AQ} = \frac{364 \times (1.767)^2}{A \times 7.30} = \frac{156}{A} = \frac{156}{14.67L}$$

14. To allow (*a*) for the dosing-tank loss (h_d) which is usually of the same magnitude as the siphon loss (h_s) and the pipe loss (h_p), and (*b*) for the use of a uniformly sloping tank wall rather than a parabolic one, vertical tank walls are required at the top, of a depth greater than the dosing tank loss. *By inspection, try* 1.33 *ft.*

15. Equating the tank capacity computed from the assumed dimensions to the tank volume obtained from the cycle time, the tank length L = 15. 42 ft. from

$$1{,}250 = \left(8.21 - 1.87 + \frac{156}{14.67L}\right) \times 14.67L - \left[(8.21 - 1.87 - 1.33) + \frac{156}{14.67L}\right] \times \frac{(14.67 - 7.0)}{2}L$$

16. The dosing tank loss for the tank dimensions chosen is

$$h_d = \frac{156}{14.67 \times 15.42} = 0.69 \text{ ft.},$$

which is approximately equal to h_s and h_p. The total computed loss is, therefore, $h_d + h_s + h_p = 2.20$, and the total difference in elevation between the discharge line of the dosing tank and the nozzle openings is 6.7 + 2.20 = 8.90 ft., which equals, in this case, the value of 8.90 ft. assumed in the basic data.

Dosing Cycle.

17. A dosing cycle of 4.5 min., under conditions of maximum sewage flow, was assumed. This is made up of the dosing time and the rest period. Although good results can be secured with continuous operation at times of maximum flow, it is well to ensure satisfactory tank and nozzle operation by providing for a minimum rest period of say 0.5 min. To estimate the probable rest period, the dosing time may be assumed to consist of (*a*) the siphon-starting time (t_a) during which the tank level drops a distance equal to the dosing-tank loss, (*b*) the time (t_b) required to empty the remaining vertical portion of the tank and (*c*) the time (t_c) required to draw down the sloping portion of the tank.

(*a*) Time (t_a) may be found from the equation proposed by the Pacific Flush Tank Co.:

$$t_a = 454\left(\frac{a_s}{Q}\right)^2 = 454\left(\frac{1.767}{7.30}\right)^2 = 26.5 \text{ sec.}$$

(*b*) Time (t_b) is given by the common formula of theoretical hydraulics for discharge under a falling head:

$$t_b = -\frac{A}{C_n a_n \sqrt{2g}} \int_{h_1}^{h_2} \frac{dh}{\sqrt{h}} = \frac{2A}{C_n a_n \sqrt{2g}} (h_1^{1/2} - h_2^{1/2})$$

where C_n is the coefficient of discharge of the nozzle field and a_n is the total area of the nozzle openings. The area of a $1\frac{3}{16}$-in. Worcester nozzle with $\frac{3}{16}$-in. spindle is 0.00340 sq. ft. The combined area $a_n = 156 \times 0.0034 = 0.530$ sq. ft. C_n may be computed approximately from the loss of head (8.21 ft.), excluding the dosing tank loss, found for the average nozzle "N" for the maximum rate of discharge, 7.30 c.f.s.

$$C_n = \frac{Q}{a_n\sqrt{2gh}} = \frac{7.30}{0.530 \times 8.02\sqrt{8.21}} = 0.60$$

$$t_b = \frac{2 \times 14.67 \times 15.42}{0.60 \times 0.530 \times 8.02}[(8.21)^{1/2} - (8.90 - 1.33)^{1/2}] = 21.2 \text{ sec.}$$

(*c*) To find time (t_c) the tank area must be expressed in terms of the head, since the area decreases with the head. The area A at any head h is

$$A = \left[7.0 + \frac{(h - 1.87)(14.67 - 7.0)}{(8.90 - 1.33 - 1.87)}\right] \times 15.42$$

$$A = 20.8\ (3.33 + h)$$

$$t_c = \frac{-20.8}{0.60 \times 0.530 \times 8.02}\int_{h_1}^{h_2} \frac{(3.33 + h)}{\sqrt{h}} dh$$

$$t_c = 8.15\,[6.66(h_1^{1/2} - h_2^{1/2}) + \tfrac{2}{3}(h_1^{3/2} - h_2^{3/2})]$$

Since $h_1 = 8.90 - 1.33 = 7.57$ and $h_2 = 1.87$,

$$t_c = 174 \text{ sec.}$$

The dosing period is, therefore, $t_a + t_b + t_c = 3.7$ min., and the resting period, $4.5 - 3.7 = 0.8$ min., which satisfies the requirements. Were this not so, it would be necessary to select different nozzles or change some of the assumptions.

Computations for Inlet Structures.—It is assumed that the inlet structures consist of air-lock feeds and weirs as shown in Fig. 202.

Air-lock Feed.

18. *Try two 6-in. feeds, 5 ft. wide.*

The feed loss (h_f) by the expression given in Section 386, for a maximum inflow rate of 3 m.g.d. = 4.64 c.f.s., becomes:

$$h_f = 0.129 \left(\frac{Q_i}{L_f}\right)^{1.64} = 0.129 \times (0.464)^{1.64} = 0.04 \text{ ft.}$$

Inlet Weir.

19. *Try a weir 15 ft. long.*

From the Francis formula, the weir loss (h_w), using a coefficient of discharge of 3.0, is found to be:

$$h_w = \left(\frac{Q}{CL_w}\right)^{2/3} = \left(\frac{4.64}{3.0 \times 15}\right)^{2/3} = 0.20 \text{ ft.}$$

The Pacific Flush Tank formula, $h_w = 0.470\ (Q_i/L_w)^{0.565}$, yields about the same result.

20. The combined loss of the inlet structures is $h_f + h_w = 0.24$ ft. for maximum flow. The sewage level in the inlet channel must, therefore, rise 0.24 ft. above the maximum level of the dosing tanks at times of highest sewage flow. If this value is unsatisfactory the inlet loss may be cut down (1) by changing the dimensions of the feeds and (2) by changing the length of the weir.

388. Final Settling Tanks.—Settling tanks which follow coarse-grained contact beds or trickling filters commonly are given a detention period of ½ to 5 hours. Any one of the common types of settling tanks may be employed, and plants have been built with horizontal-flow, flat-bottomed or hopper-bottomed tanks; with vertical-flow tanks of Dortmund type; and with tanks equipped with sludge-removing mechanisms. The principles of design do not differ materially from those applicable to preliminary sedimentation tanks.

The quantity of sludge deposited in trickling-filter humus tanks varies from 300 to 800 gal. per million gallons of filter effluent, averaging about 500 gal. The water content averages about 92.5 per cent, varying from 90 to 95 per cent. Sludge is generally withdrawn weekly and storage space must be provided accordingly. In some plants longer intervals elapse between cleanings and sludge is removed only when septicization or excessive accumulation of solids interferes with clarification. During the unloading periods more frequent cleaning is essential.

The flocculent character of the sludge suggests the use of vertical-flow tanks of the Dortmund type and these have been

used widely. The design characteristics of the final tanks at Gloversville, N. Y., Fitchburg, Mass. and Akron, O., are given in Table 96.

Sludge from humus tanks may be air-dried on sludge-drying beds, but is more commonly pumped to digestion tanks where these are provided in connection with preliminary sedimentation.

389. Examples of Filter Design.—A few examples of the design values employed in existing works are given in Tables 94 to 96. These illustrate typical sewage filters of the three different types in common use in the United States.

Intermittent Filters.—The three different stages in development of this type of filter are exemplified by the installations at (1) Concord, Mass., which treat raw sewage; (2) Marlborough, Mass., which deal with settled sewage; and (3) Alliance, Ohio, which follow contact beds.

TABLE 94.—COMPARISON OF INTERMITTENT FILTERS

	Concord	Marlborough	Alliance[1]
Date of construction	1899, 1928	1891, 1908–11	1913, 1917
Tributary population	2,400	15,080	20,000
Population designed for	3,050	14,000	
Volume of sewage, m.g.d.			
Maximum rate of flow	0.69	3.25	4.50
Average rate of flow	0.42	1.18	2.62
Minimum rate of flow	0.27	0.33	1.55
Number of beds	7	33	3
Average depth, ft	....	4.5–6	2.2–3
Area, acres	6.3	20.9	3
Character of filtering material	Sand	Sand	Cinders and sand
Effective size of material, mm	0.06–0.50	0.07–0.20	
Dosing apparatus	Pump	18-in. siphon	
Method of distribution	2 to 4 inlets per bed	1 to 4 inlets per bed	Wooden troughs
Underdrains	None		
Size, in	....	5	6–8
Spacing, ft	....	20–50	38–40 (2 beds) 9 (1 bed)
Average loading			
Million gal. per acre per day	0.07	0.06	0.87
Population per acre			
Tributary	381	721	6,670
Design	485	670	

[1] This plant was replaced in 1929 by Imhoff tanks, trickling filters and humus tanks.

Contact Beds.—The following two installations are chosen for illustration, (1) Alliance, O., and (2) Madison-Chatham, N. J.

The beds at Alliance are of the single-contact variety; those at Madison-Chatham provide for double contact.

TABLE 95.—COMPARISON OF CONTACT BEDS

	Alliance[1]	Madison-Chatham
Date of construction	1910, 1917	1911
Tributary population	20,000	
Population designed for	....	7,000
Volume of sewage, m.g.d.		
Maximum rate of flow	4.5	2.0
Average rate of flow	2.6	0.4
Minimum rate of flow	1.6	0.15
Preliminary treatment of sewage	Rack and sedimentation	Racks and Imhoff tanks
Total area of contact beds, acres	3	1
Area of primary beds, acres	3	0.5
Area of secondary beds, acres	None	0.5
Character of filtering material	Cinders and slag	Stone
Size of material, in	Cinders, ⅛–¾ Slag, ¾–1½	1–2.5
Number of contact beds	12	8
Dimensions, ft.		
Length	....	75
Width	....	75
Depth	4–4.5	4.2
Dosing apparatus	3 siphons	"Ansonia" apparatus
Dosing period, hr.		
Filling	2½–3	
Contact	⅛	
Emptying	2½–3	
Type of distributors	Ducts filled with gravel	Beds dosed from below
Underdrains		
Type	Tile	Split tile
Size, in	3	6
Spacing, ft	11	0.67 center to center
Average loading		
Million gal. per acre per day	0.87	0.40
Million gal. per acre-foot per day	0.19–0.22	0.10
Population per acre	6,670	7,000[2]
Population per acre-foot	1,480–1,670	1,670[2]
Further treatment of effluent	Intermittent filters	Humus tanks and sand filters

[1] This plant was replaced in 1929 by Imhoff tanks, trickling filters and humus tanks.
[2] Provided for in design.

Trickling Filters.—Data for three installations with which the authors were connected, namely those at Gloversville, N. Y., Fitchburg, Mass., and Akron, O., are tabulated below.

TABLE 96.—COMPARISON OF TRICKLING FILTERS

	Gloversville	Fitchburg	Akron
Date of construction	1911	1914	1927
Population designed for	20,000	40,000	260,000
Capacity designed for, m.g.d	3	4.0	33
Average sewage flow, m.g.d	2.6	3.8	
Area of filters, acres	3.1	2.1	13.7
Character of filtering material	Limestone	Crushed granite and trap rock	Hard limestone
Size of material, in	1–2½	1–2	1–2
Average depth of material, ft	5	10–10.5	10
Method of distribution	Taylor square-spray nozzle	Worcester circular-spray nozzle	Circular-spray nozzle
Size of orifice of nozzle, in	¾	13/16	0.906
Distance of orifice above surface, in	8	Flush	2
Distance center to center of nozzles, ft	12	15	11
Number of nozzles per acre	303	230	300
Head on nozzles, ft	6.3	8.5	5.93–1.39
Character of floor system	6-in. slotted channel pipe on 3-in. concrete floor	Grooved concrete floor blocks	Vitrified floor blocks on vit. channel blocks on concrete floor
Preliminary treatment of sewage	Racks, grit cham., fine screens and settling	Racks, grit chambers and Imhoff tanks	Racks, detritus tanks and Imhoff tanks
Further treatment of effluent	Settling and sand filtration	Humus tanks	Humus tanks
Average filter loading			
Million gal. per acre per day	0.84	1.84	2.4[1]
Persons per acre	7,100	19,300	18,930[1]
Persons per acre-foot	1,420	1,890	1,893[1]
Dosing tanks			
Number	1	2	14
Capacity per unit, gal	16,700	40,000	23,000
Size of siphon, in	30	30	36
Volume of dose, gal. per acre	8,100	20,000	11,700
Dosing time, min	3½	5	2
Secondary sedimentation tanks			
Number	2	4	12
Inside dimensions, ft	36 (diameter)	30 (diameter)	65 × 50
Maximum water depth, ft	33.2	22.5	7.5
Total water capacity, cu. ft. per capita	1.38	1.17	1.22[1]
Average detention period, hours	2.12	2.21	1.72[1]
Slope of tank bottom	1 on 1	1 on 1½	4%
Sludge removed by	Gravity	Gravity and pump	Flushing and gravity

[1] Provided for in design.

Literature

SANDS, E. E. Hydraulics of Dosing Tanks and Trickling Filters. *Eng. News-Rec.*, 1922; **89**, 67.

Buswell, A. M., and S. I. Strickhouser. 1929. The Depth of Sewage Filters and the Degree of Purification. Bull. 26, Illinois State Water Survey. Urbana, Ill.

Problems

1. During 1927 the trickling filters at Fitchburg, Mass., (estimated population 40,000) received 1,379.08 m.g. of sewage in 364 days. The area of the filters is 2.07 acres and they are 10 ft. deep. The sewage applied to the filters contained on an average 12.61 p.p.m. of nitrogen as free ammonia and 2.96 p.p.m. of albuminoid nitrogen. Find the filter loading per unit of area and volume in gallons per day, persons, and grams and pounds of nitrogen. *Ans.* 1,830,000 gal./day per acre; 183,000 gal./day per acre-foot; 19,300 persons per acre; 1930 persons per acre-foot; 103,000 lb./acre per year (N); 10,300 lb. per acre-foot per year; 128,000 gm./acre per day; 12,800 gm. per acre-foot per day.

2. Sand of effective size 0.15 mm. is available for use in intermittent sand filters. If a half-acre square bed, composed of sand 4 ft. in depth, is dosed at a rate of 60,000 gallons per acre per day (*a*) how much sewage should be applied per filling; (*b*) what will be the time elapsing between fillings, (*c*) the rate of filling, (*d*) and the time required for filling; (*e*) how long will the bed rest between fillings if the temperature of the sewage is 54°F., assuming that the underdrains have a free outlet (estimate on a basis of $C = 200$ in Hazen's formula); (*f*) to what height will the sand hold water by capillarity? *Ans.* (*a*) 40,800 gal. (*c*) 4.4 c.f.s. (*e*)
(*b*) 32.6 hours. (*d*) 21 min. (*f*) 2.6 in.

3. 16 half-acre sand beds designed to handle the sewage from 10,000 people after clarification by Imhoff tanks, are arranged in groups of 4, each group being supplied by a main distributor pipe. Dosing of all 4 groups is controlled by one automatic dosing tank holding 1,545 gallons per foot of depth. The tank has a maximum drawing depth of 4.62 ft., and the center of the outlet pipe is 18 in. below low-water level in the tank. Sewage is directed on to the individual beds by operating gates on the main distributor pipes. The siphon is 12 in. in diameter. Assuming a per capita sewage flow of 100 gal. per day, estimate the following:

(*a*) Inches of sewage applied to each bed per dose.

(*b*) Number of doses per bed before sewage must be diverted on to another bed.

(*c*) Characteristics of dosing cycle. *Ans.* (*a*) 0.6 in.
(*b*) 5.

4. Four contact beds, each 0.25 acre in area and 4 ft. deep, are to treat 400,000 gal. of settled sewage per day. Estimate the dosing cycle on the basis of 30 per cent voids.

5. In accordance with the principles outlined in Section 387 design the nozzle field, distribution system and twin dosing tanks for a trickling filter 10 ft. deep treating 0.5 m.g.d. of settled sewage, to meet the following conditions: (*a*) maximum rate of flow, 1.5 m.g.d., (*b*) minimum rate of flow, 0.25 m.g.d., (*c*) loading of filter, 2.5 m.g.d. per acre, (*d*) elevation of maximum discharge line above nozzles, 7.2 ft., (*e*) type of nozzle, $1\frac{3}{16}$-in. Worcester nozzle with circular spray.

CHAPTER XVIII

THE ACTIVATED-SLUDGE PROCESS

390. Development of the Activated-sludge Process.—The aeration of sewage in tanks, to hasten oxidation of the organic matter, was investigated as early as 1882 by Dr. Angus Smith, who reported on it to the Local Government Board. It was studied subsequently by a number of investigators and in 1910 Black and Phelps reported that a considerable reduction in putrescibility could be secured by forcing air into sewage in basins. Following this, experiments by Clark and Gage at the Lawrence Experiment Station, conducted during 1912 and 1913 on sewage in bottles and in tanks partially filled with roofing slate spaced about 1 in. apart, showed that in aerated sewage growths of organisms could be cultivated which would greatly increase the degree of purification obtained.

The results of the work at Lawrence were so striking that a knowledge of them led Fowler to suggest experiments along similar lines at Manchester, England, where Ardern and Lockett carried out valuable researches upon this subject. During the course of their experiments Ardern and Lockett found that the sludge played an important part in the results obtained by aeration, as announced in their paper of May 3, 1914, before the Manchester Section of the Society of Chemical Industry. At the outset it was necessary to aerate the sewage samples continuously for 5 weeks before complete nitrification was obtained. By repeatedly drawing off the clarified sewage and adding fresh raw sewage to the old sludge left in the experimental tank, the time for oxidation, however, was reduced to 24 hours and eventually to a few hours only. The sludge accumulating in this manner and inducing such active nitrification was called "activated sludge." Hatton at Milwaukee, Frank at Baltimore, and others, finally showed that the process could be operated on a practical, continuous basis by running sewage through aeration tanks, the activated sludge being mingled with the entering sewage, and later separated from it after its passage through the tanks. More recent developments in the activated sludge

process have dealt with the perfection of the control of aeration and the introduction of mechanical devices for aerating the sewage in place of compressed air. This variation of the process is called "bio-aeration."

Closely related to the activated-sludge process and originating from the same early investigations, but differing in many respects from the process as now employed, is "contact aeration", developed by Buswell, Bach and Imhoff (1925, 1926). In this process aerated contact surfaces for the support of growths of organisms are provided in the flowing-through chamber of single- or two-story tanks. These growths and surfaces apparently take the place of the activated sludge floc, at least in some measure.

391. Classification of Activation Methods.—It is evident from the preceding section that several different methods of aeration and activation have come into use. These may be classified as follows:

1. *Diffused-air Aeration, or the Activated-sludge Process Proper.*—In this method of treatment air is blown into sewage as it flows through tanks, activated sludge being added to the incoming sewage and settled out, in sedimentation basins, from the tank effluent. A part of the sludge produced is fed into the influent as "return sludge," and a part is disposed of as "excess sludge" (Fig. 206).

2. *Mechanical Aeration or Bio-aeration.*—Mechanical apparatus is employed in this method, to aerate the sewage and keep the tank contents in circulation. Absorption of air takes place from the atmosphere at the surface of the sewage. Activated sludge is added to the influent and removed from the effluent as under (1) (Fig. 207).

3. *Combined Use of Diffused Air and Mechanical Aeration.*—Air in reduced volume is blown into the sewage while circulation is maintained by mechanical means. Addition and removal of activated sludge are provided for as under (1) (Fig. 208).

4. *Contact Aeration.*—In this method of treatment,—which, strictly speaking, cannot be classified as an activated-sludge process and which will, therefore, be discussed in a separate section of this chapter,—contact surfaces in the form of laths or similar materials are submerged in the sewage, aerobic conditions are maintained and the sewage generally is forced to circulate through the contact units by the admission of compressed air from below. From time to time excessive growths are removed by increased aeration. The suspended matter in the effluent is

deposited in settling basins. Sludge is not added to the influent (Fig. 212).

Of the methods of activation, the one most widely used at present (1929) in the United States is diffused-air aeration. Mechanical aeration, which has been more widely employed in England, has only recently been used in a full-sized plant, and the remaining two methods, although successful in Germany, are still in the experimental stage in this country.

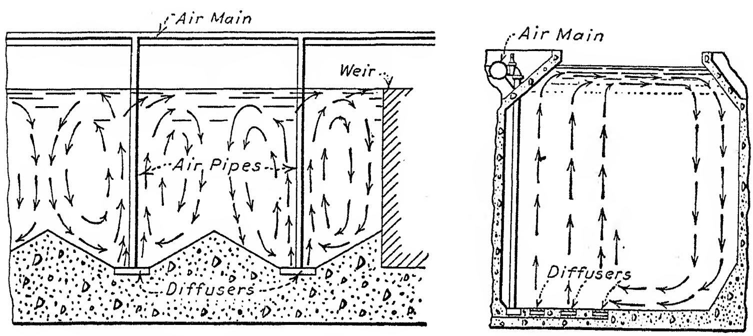
PRINCIPLES OF OPERATION

392. Aeration Units Employing Diffused Air.—In tanks of this type the air should perform three distinct functions which may be described as follows: (1) it should provide the oxygen necessary to maintain aerobic conditions and promote the growth of oxidizing organisms, chiefly bacteria; (2) it should cause the activated sludge to move through the sewage and provide contact between the gelatinous surfaces of the sludge and the putrescible matters contained in the sewage; (3) it should prevent deposition of sludge, particularly on the tank bottom where sludge accumulations quickly become septic and thus adversely affect the process. In bio-aeration units the first of these functions of the air is secured by inducing absorption of air at the sewage surface, the last two functions being performed by mechanical agitation of the tank contents.

The three functions of the air are fulfilled in two common types of aeration units which are known respectively as (1) "ridge-and-furrow" tanks and (2) "spiral-flow" tanks. In both types air is blown into the flowing sewage from *diffusers* placed at the bottom of the tanks. These diffusers are commonly porous plates through which the air passes into the sewage in the form of fine bubbles, thus presenting the greatest surface area possible for contact between air and sewage.

In ridge-and-furrow tanks (Fig. 206*a*) the diffusers are situated at the bottom of depressions between ridged spacers. Surfaces upon which the sludge might settle are thus avoided. Generally the diffusers are placed in rows across the tanks at right angles to the direction of flow, thus causing the sewage to be subjected to successive impacts of air, sometimes referred to as "air baffles." Continuous flow is provided, intermittent operation as originally practiced at Manchester, England, never have been adopted on a large scale in this country.

In spiral-flow tanks (Fig. 206*b*) the diffusers are placed in rows running the length of one side of the tanks. The air, tending to rise vertically along this one side, induces a rotation of the sewage in the tank, which combines with the longitudinal displacement of the sewage to produce helical motion, carrying a certain amount of air across the tank and downward. This type of flow successfully prevents deposition of the sludge, ensures a longer path of travel of both air bubbles and sewage, and permits of greater absorption of atmospheric oxygen at the sewage sur-



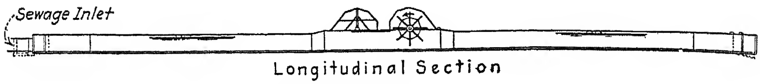
a.—Ridge-and-furrow tank. *b*.—Spiral-flow tank.

FIG. 206.—Aeration units employing diffused air.

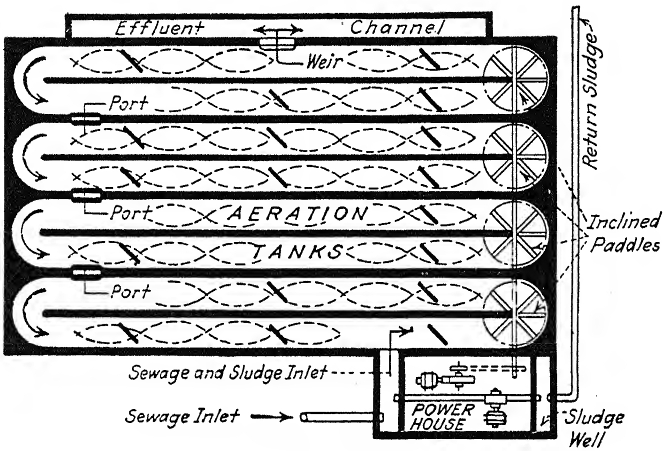
face. At the same time it reduces short-circuiting. Deflectors at the sewage surface and rounding of corners aid in establishing and maintaining spiral motion. At Indianapolis the horizontal path of the sewage is but 952 ft. in tanks 15 ft. deep and 20 ft. wide, but since the pitch of the helix is about 1.75 ft. the actual length of travel of the sewage is estimated to be over 7 miles.

The advantages of this type of arrangement have led to its being adopted widely. In England transverse baffles with an ample opening, commonly placed on the same side of the tank as the diffusers, are often employed to effect further reduction in short-circuiting, which may be associated with the formation in the center of the spiral of a core in which motion is only horizontal and longitudinal.

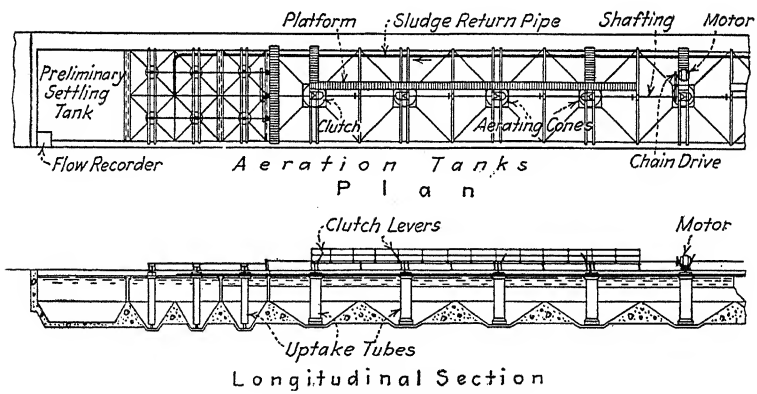
393. Aeration Units Employing Mechanical Agitation (Bioaeration).—The fact that but a small portion of the oxygen



a.—Sheffield system.



b.—Hartley system.



c.—Simplex system.

FIG. 207.—Aeration units employing mechanical agitation.

contained in the air blown into the sewage is actually utilized (2 to 10 per cent), and that one of the important functions of the air is to keep the sludge in suspension, was utilized first by Haworth at Sheffield to develop a mechanical device which would perform the same functions as the air, by permitting absorption of oxygen from the atmosphere in contact with the sewage surface, while inducing a sufficient velocity of the sewage in the tank to keep the sludge suspended and moving through the liquid.

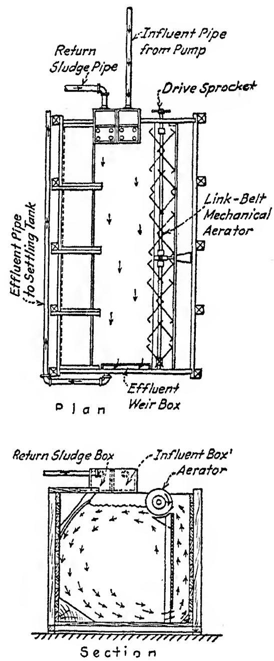


FIG. 207*d*.—Link-Belt system.

Of the devices which have been employed in full-sized bioaeration units, the following may be taken as illustrating somewhat different principles of design and operation:

1. The Haworth or Sheffield system.
2. The Hartley system.
3. The Bolton or Simplex system.
4. The Link-Belt system.

Numerous other devices have been suggested, and experimental studies of some of them have been promising. Since it is impossible to describe them all in the space available, the student is referred to current engineering literature for their description.

The Haworth or Sheffield System.—As employed at Sheffield, England (Fig. 207*a*), the aeration tank is divided by walls or round-the-end baffles, into a series of long and relatively narrow and shallow channels, giving a length of travel of about a mile. The outlet channel feeds a transverse channel from which the sewage overflows into settling tanks. A return channel leads from this transverse channel to the inlet end and returns a small portion of the tank effluent to the incoming sewage, somewhat reducing the volume of return sludge pumped.

Aeration takes place at the sewage surface, new surfaces of liquid being brought into contact with the air and circulation of the sewage being maintained by two rows of paddles operated from horizontal shafts crossing the channels about midway of their length. The paddles are staggered and consist of two sets of

radial arms forming a rimless wheel. Short waves are created at the sewage surface. The ends of the channels are rounded and the dividing walls terminate in pear-shaped enlargements. This feature prevents deposition of sludge at this critical point and preserves the waves (see Section 392).

The Hartley System.—In this system, employed at Birmingham, England, paddle wheels mounted on shafts, which are inclined at a small angle from the vertical, are provided at the return bends of long, narrow and shallow channels (Fig. 207*b*). The channels are endless, as at Sheffield, but are broken up into several units operated in series. The straight part of the channel contains a number of baffle plates which can be moved diagonally to the flow to cause spiral motion, which also is induced in the sewage by the inclination of the paddle wheels. This motion is said to aid surface aeration and keep the sludge in suspension at lower velocities (see Section 392).

The Bolton or Simplex System.—Relatively deep, hopper-bottom tanks are employed in this system (Fig. 207*c*). Each tank unit is provided with a vertical tube, at the top of which a revolving cone provided with vanes is suspended. The tubes are placed in the centers of the units, a small clearance being provided at the bottom, through which sewage and sludge enter. As the cone spins the sewage is drawn into the tube, passes upward and on reaching the top is thrown out in a thin sheet over the tank surface. Aeration is thus secured and the sewage circulates, moving upward inside the tube and downward outside. The tank contents are circulated two or three times per hour. This system is employed at a number of places in England and at Princeton, Illinois, and Rochester, Minn. The tanks are sometimes operated in parallel, sometimes in series.

The Link-Belt System.—The Link-Belt aerator shown in Fig. 207*d* induces surface aeration and spiral (helical) motion by means of a paddle mechanism similar in design to a ribbon conveyor. This mechanism, operating partially submerged on a horizontal shaft running the length of the tank, is mounted above a longitudinal partition wall set near one side of the tank. Rotation of the paddle mechanism causes the sewage to rise in the narrow division partitioned off from the main tank, and induces a return current in the wider portion of the tank. Spiral flow is thus established. At the same time the surface of the sewage is agitated and aeration of the sewage takes place. In the wide

portion of the tank transverse baffles, under which the sewage must flow, prevent short-circuiting.

394. Aeration Units Employing both Diffused Air and Mechanical Agitation.—Greater utilization of the oxygen contained in the diffused air blown into sewage has been obtained by Imhoff at Essen-Rellinghausen, Germany, by providing for mechanical agitation as well as diffused-air aeration (Fig. 208). Using a submerged paddle mechanism revolving on a horizontal axis running the length of the aeration tank, as well as diffuser plates placed in much the same position as in spiral-flow tanks, Imhoff has secured a utilization of 30 per cent of the oxygen injected through the diffusers. He ascribes this chiefly to the fact that the movement of the paddles is opposite in direction to that of the air and that the rising air is thus forced to take a long path through the sewage. Lower power consumption also is claimed for this arrangement. Previous experimental attempts, by other workers, to combine the use of diffused air with mechanical agitation were generally less promising, except in the treatment of industrial wastes.

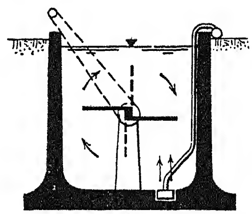


FIG. 208.—Aeration unit employing both diffused air and mechanical agitation.

395. Need for Preliminary Treatment.—Generally grit and other heavy material must be removed from sewage which is to be treated by the activated-sludge process. These substances are not readily maintained in suspension and tend to weigh down the sludge and cause it to settle. Their removal reduces accumulation and septic decomposition of sludge on the tank bottoms, and also reduces the clogging of diffuser plates. It is best to keep solids which are large in bulk, and floating matters, too, out of aeration units. They do not respond readily to treatment, tend to settle on the bottom or float in unsightly collections on the surface, and affect adversely the quality of the sludge settled from the tank effluent. Oil and grease seem to interfere seriously with aeration in all types of activated-sludge units.

Aside from the removal of those matters which interfere with the proper operation of the treatment process, it may be desirable to remove the readily settling solids from the sewage prior to aeration in order to secure (1) a reduction in air requirements (from 15 to 20 per cent reduction has been recorded) or in the period of aeration, and (2) a reduction in the bulk of the sludge

to be disposed of, fresh and digested solids giving about a 95- and 80-per cent sludge respectively, while activated solids produce about a 98-per cent sludge. Sometimes the lower density of activated sludge produced from clarified sewage is disadvantageous, and may cause greater difficulty of dewatering the excess sludge. English experience seems to show that these troubles may be avoided if aeration of clarified sewage is carried to a point where the effluent is well nitrified, while with raw sewage a well-clarified effluent is commonly sufficient. Where sludge of high nitrogen content is obtained and marketed as fertilizer it may be advantageous to dispense with pre-sedimentation. As a general rule, however, it seems unnecessary to produce more activated sludge than is required to carry on the treatment process.

Choice of pretreatment methods depends upon local conditions. Grit chambers, racks, fine screens, skimming tanks and single- or two-story settling tanks all have their part in the economy of the process.

396. Need for Final Treatment.—The effluent from aeration tanks requires sedimentation in order to produce a clear effluent with low biochemical oxygen demand. In all systems of operation part of the sludge is discharged into the influent. As shown in Section 401 the return sludge may, under certain circumstances, receive further aeration, known as "reactivation."

Vertical-flow tanks of the Dortmund type and horizontal-flow, mechanically-cleaned tanks have been employed most commonly as sedimentation units.

One of the major problems of the process is the disposal of the excess activated sludge. This problem looms large because of the great bulk of the sludge produced and its unstable character. For these reasons digestion of the sludge, either alone or together with fresh sewage solids from pretreatment tanks, has come into use. A dense sludge similar to Imhoff sludge results from digestion of mixed sludge, while a more watery sludge (about 95 per cent) seems to be produced in the absence of fresh solids.

Where the sewage is only partially treated by the activated sludge process (Section 405) the effluent after settling may be oxidized further by use of trickling filters.

397. Establishment of Sludge Activity.—As with other biological sewage treatment methods, some time will elapse before effective operation of new aeration units is established. This

is because of the necessity of building up the desired quality of activated sludge, the vehicle of the purifying agencies.

The production of activated sludge at new works, or at a plant which has been out of operation for some time, is accomplished without delay if good activated sludge can be obtained from a nearby treatment plant in sufficient quantity to start operations. Next best perhaps is the introduction of sludge from trickling-filter humus tanks, for this type of sludge is similar in character to activated sludge, and has been found to aid in the accumulation of an adequate supply of activated sludge from sedimentation and aeration of fresh sewage. For example, in starting some of his activated sludge experiments at the Milwaukee testing station, Copeland produced good sludge in this manner in a little more than a week.

When neither activated sludge nor trickling-filter humus are available, the aeration unit may be broken in by operating it on a fill-and-draw basis, adding fresh sewage about four times a day, or by working the tanks at a greatly reduced rate and discharging into the influent all of the sludge settling from the tank effluent. Normal operation will become possible in 2 to 4 weeks.

Once the tank is broken in, the sludge will retain its action under favorable conditions, with continuous accretion in amount, thus creating excess sludge, and treatment can be continued indefinitely. It is essential, however, not to overwork the sludge, otherwise it will gradually lose its activity and become ineffective.

Experiments with artificial sludges, such as iron, aluminum and manganese hydroxides and silica "gels," have shown that they are of little practical benefit. Although they may intensify oxidation, it is said to be difficult to maintain them "active." Ferrous iron and nitrites were found by Buswell to act as carriers of oxygen to the bacteria, taking up oxygen quickly themselves.

398. Properties of Activated Sludge.—Thoroughly activated sludge floc generally has a golden-brown color and is relatively compact. Under-aerated floc is usually light-brown in color, fluffy and relatively light in weight. Over-aerated floc may be of a muddy-brown color, probably due to unabsorbed colloidal substances, and may be considerably broken up and disintegrated into relatively fine material which settles slowly and leaves a turbid supernatant liquid. A trained operator often relies upon the color of the sludge as an indicator of its quality.

Well-activated floc settles more readily than under-aerated floc and forms a denser sludge, leaving the supernatant liquor clear. Under-aerated floc is so light that it forms a very thin and bulky sludge which is apt to be carried out of sedimentation tanks with the effluent.

Activated sludge contains a very small proportion of the suspended solids in the sewage, as compared with sludge obtained from the treatment of sewage by other processes. When well aerated and accumulated under favorable conditions, it generally contains about 2 per cent solids (98 per cent water). Particularly well-activated sludge may contain as much as 3 per cent solids, or slightly more, while under-aerated sludge may contain only 0.5 per cent solids, or even less. This variation in density has a very important bearing upon the design and operation of the plant, for a sludge containing 0.5 per cent solids has a volume four times as great as that containing 2 per cent solids, as explained in Section 322.

Occasionally a phenomenon known as "sludge bulking" is observed in treatment plants. It is so called because the volume of the sludge becomes unusually great. An unsatisfactory degree of purification usually results. Bulking seems to be associated with septic conditions, which may be due to factors such as under-aeration, prolonged detention in settling tanks, accumulation of sludge on the bottom of aeration units or sludge-return channels, sudden discharge of septic solids from the sewer system, and introduction of objectionable industrial wastes. An increase in the protozoal life of the sludge is commonly noted when bulking occurs. Bulking seems to be more common with strong English sewages than with the more dilute American sewages. This may be due to the greater amount of oxygen in the liquid, especially in the storm water, and to the more generous use of air, in this country.

More or less bacterial digestion of the organic matter of the sludge goes on continuously in aeration and sedimentation tanks. This action is more marked at moderate temperatures, when bacterial activity is high, than at the low temperatures of winter.

In certain tests made by the authors it was found that aeration can be carried so far that the sludge will become disintegrated and lose its clarifying power. At such times the increase in the proportion of sludge accumulating in the tank becomes small, and there may even be a decrease. This condition can be remedied

by removing a portion of the sludge, by reducing the period of aeration, or both; this will result in the rapid building up of a large quantity of fresh activated sludge and, almost immediately, in a well-clarified effluent.

399. Measurement of Sludge.—The volume and character of sludge is such an important matter in this process of treatment that its proper measurement is essential. The volume of sludge contained in the aeration tank liquor may be determined by allowing a measured portion of the liquor to settle for a definite period of time and then reading the volume of the settled sludge. If comparisons are to be made of the quantities of sludge produced and used it is necessary that measurements be made under

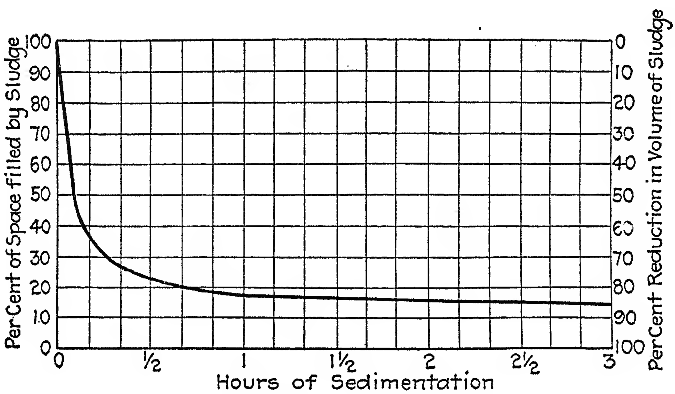


FIG. 209.—Rate of sedimentation of activated sludge (Milwaukee).

reasonably comparable conditions. One of the most convenient means of measurement is a 1,000-cc. graduated glass cylinder about 2¼ in. in diameter and 16 in. high. By its use the percentage of sludge can be read directly. For determining and regulating the proportion of activated sludge present in the aeration tanks and sedimentation tanks, a period of sedimentation in the measuring glass of 1 hour is commonly used. At Milwaukee a period of ½ hour proved to be adequate, and its use was advantageous as the results of tests could be obtained with relative promptness. Shorter periods of settling are subject to too great error and variation.

It has been found at Milwaukee that a test of this kind, made at the same hour each morning, serves as a valuable means of regulating and controlling the process, the adjustment of the

sludge being accomplished by fixing the rate of drawing off sludge for disposal, according to the indications of the test.

An occasional series of tests for several different sedimentation periods may furnish valuable information and enable the operator to compare his operating conditions with those of other places where different test periods are adopted. The rate of sedimentation, or the volume of sludge after different periods of settling at Milwaukee, is shown by Fig. 209.

A laboratory centrifuge also can be employed to advantage in order to obtain immediate readings. Sludge measurements of greater exactness may be obtained by filtering a known volume through paper or asbestos, and drying and weighing the sludge collected.

400. Volume of Sludge Produced.—As indicated in Section 395, limited aeration of raw sewage with activated sludge produces a clear effluent after sedimentation, and the quantity of sludge is probably as great as, or greater than, that yielded by carrying treatment to a point where a thoroughly nitrified effluent is produced. The amount of sludge depends primarily upon the quantity of suspended and colloidal matter in the sewage, which varies considerably, particularly when the sewage is subjected to preliminary clarification or when it contains storm water or certain classes of industrial wastes, such as iron salts and organic matter from packing houses and tanneries.

Experience indicates that a short period of aeration with good activated sludge will reduce the suspended matter carried by the influent about 90 per cent. A portion of the suspended matter thus removed is converted into gaseous and dissolved substances by digestion, the remainder forming sludge. The proportion of sludge thus digested is subject to considerable doubt; values as high as 50 per cent and as low as 4 per cent have been reported; the lower values are more probable under average conditions of operation.

The average quantities of sludge produced at different activated-sludge plants are shown in Table 97.

The water content of the sludge produced is extremely important because of the feature of discharging sludge into the influent. Furthermore, where activated sludge is to be disposed of without first subjecting it to digestion, the operation of the process must be conducted with a view to producing a sludge that is small in bulk, that can be dried readily and, if the sludge is to be marketed,

a sludge that is high in valuable fertilizing constituents, especially nitrogen.

TABLE 97.— QUANTITIES OF SLUDGE PRODUCED IN THE ACTIVATED-SLUDGE PROCESS

City	Year	Gal. of sludge per m.g. of sewage	Per cent water in sludge	Specific gravity of sludge	Lb. of dry solids per m.g. of sewage
Chicago, Ill. (Des Plaines)......	1924	5,100	98.5	1.02	650
Indianapolis, Ind..............	1927	13,700	98.3	1.00[1]	1,950
Gastonia, N. C................	1922	13,000	98		
Pasadena, Cal.................	1928	32,100	99.1	1.004	2,420
Milwaukee, Wis...............	....	15,000	98.5–99	.....	2,000 ±

[1] Assumed in reporting dry solids.

401. Reactivation.—If the period of aeration of the sewage, the proportion of activated sludge discharged into the influent, and all other operating conditions are suitably regulated, the sludge can be kept continuously in good active condition. On the other hand, if the sewage is under- or over-aerated, or if the activated sludge is overworked, it will lose a portion or all of its activity and must be reactivated in some manner, in order to restore it to a condition in which it can exercise its proper functions efficiently. Reactivation may be carried out in the mixture of sewage and sludge, or the sludge, after separation from the major portion of the treated sewage in sedimentation tanks, may be removed to separate tanks and there aerated for a period of time to restore its normal activity.

In some cases reactivation of the sludge has been advantageous as a regular part of the process, permitting the use of a smaller quantity of air and volume of return sludge than where the process was conducted so as to maintain the sludge in prime condition throughout.

In other cases there has appeared to be little if any advantage in reaeration of the sludge in separate tanks. In tests at Milwaukee, where sludge containing from 2 to 3 per cent of solids was aerated, the air appeared to accumulate in large bubbles and escape from the surface at irregular intervals. This tendency appeared to render the air less effective as a means of agitation and possibly also to interfere to some extent with the uniform distribution of dissolved oxygen throughout the sludge.

On the whole, American practice seems to tend toward maintaining the sludge in good condition without reaeration, by providing for adequate oxygenation in the tanks. Where only partial treatment by activated sludge is employed, however, reaeration appears to be almost always a necessary feature of the process (see Section 405).

402. Degree of Purification Secured.—An important characteristic of the activated sludge process is the high degree of purification which may be obtained.

So large a proportion of the suspended solids and colloids may be removed that under ordinary conditions the settled effluent will be clear and contain but little suspended matter and comparatively few bacteria. The reduction in bacteria, however, is accomplished by physically removing them, rather than by destroying them as in the case of disinfection. As far as pathogenic organisms are concerned, actual destruction probably takes place, due chiefly to the presence of predatory protozoa, food rivalry of other bacteria and the time factor.

In many cases color may be removed, probably by action similar to that of mordants.

Odors may be oxidized directly or absorbed, the effluent being comparatively free from odors noticeable to the ordinary observer.

Oxidation of organic matter in sludge occurs coincidentally with the flocculation, and there may be nitrification where the treatment is fairly complete. In the latter case, the treated sewage may be charged with a substantial quantity of available oxygen in the form of nitrites or nitrates, which will serve as a factor of safety against future putrefaction.

As early as 1917 Hatton determined upon the following standard as a measure and limit of the improvement to be effected by the activated sludge process at Milwaukee:

Reduction of bacteria, per cent	90
Reduction of suspended solids, per cent	95
Stability by methylene blue, hours	72

The results obtained at a number of activated-sludge plants are given in Table 98.

403. Winter Operation.—In very cold weather at Milwaukee, Copeland found that the free ammonia was not reduced and nitrates were not formed in material quantity, but the removal

TABLE 98.—RESULTS OF ANALYSES OF SEWAGE AND EFFLUENT FROM ACTIVATED-SLUDGE PLANTS
(Results in p.p.m.)

	Houston, Texas		Indianapolis, Ind.	Milwaukee, Wis.	Pasadena, Cal.	San Marcos, Tex.	Sherman, Tex.
	North Side Plant	South Side Plant					
Year	1923	1923	1927	1927	1928–29	1920	1920
Period covered by analyses	1 year	1 year	1 year	1 year	6 months	10 days	11 days
Total solids							
Influent	.....	.....	.....	1,000			
Effluent	.....	.....	.....	750			
Per cent removed	.....	.....	.....	25			
Suspended solids							
Influent	272	166	318	250–300	336	42	264
Effluent	34.3	9.1	27	25	14	3	73
Per cent removed	87.5	94.5	91.5	90–92	95.8	92.9	72.4
Oxygen consumed							
Influent	.....	.....	.....	....	.....	18	64
Effluent	.....	.....	.....	....	.....	8.5	32.0
Per cent removed	.....	.....	.....	....	.....	52.8	50.0
Biochemical oxygen demand (5-day)							
Influent	120	.....	236	200–250	207	57	165
Effluent	8.2	.....	31.6	6 or less	8.7	16.0	33.0
Per cent removed	93.1	.....	86.5	97 or more	95.8	71.9	80.0
Dissolved oxygen							
Influent	0	.....	.....	....	.....	2.8	2.5
Effluent	3.9	.....	5.5	....	3.9	7.6	6.2
Nitrogen as							
Free ammonia							
Influent	19.2	16.5	.....	....	18.4	17.	26
Effluent	7.2	5.0	.....	....	8.4	2.8	23.0
Per cent removed	62.5	69.7	.....	....	54.4	83.6	11.5
Nitrates in effluent	4.1	11.3	0.7	4.0	4.5	8.8	1.2

of organic matter was so great and the low temperature of the sewage permitted the liquid to absorb so much dissolved oxygen that the effluent was stable in the absence of nitrates. It was also found that melted snow from the streets and the cold muddy storm water in the early spring caused a decrease in the size of the floc and in its ability to absorb colloidal matter. This condition could be limited, in part at least, by increasing the air supply about 25 per cent.

Cold weather was found by Pratt and Gascoigne to have no appreciable effect on the process at the Cleveland Testing Station, where clarification instead of stability was the operating aim. Lederer has suggested that in cold weather the turbidity of the effluent is a good index of its quality. At such times he found the stability (Section 256) was 100 per cent with turbidities of 10 p.p.m. or less; it varied between 50 and 100 per cent with a turbidity of 15; and fell off rapidly when the turbidity exceeded 15.

Generally speaking, temperatures below 50°F. have been shown to exert a marked retarding effect upon oxidation, while temperatures above 90°F. cause an initial lag in nitrification with a subsequent increase in the rate.

404. Effect of Industrial Wastes.—In the activated-sludge process, as in other treatment processes, the presence of industrial wastes may have an appreciable bearing upon the operation and design of the works. In activated-sludge plants any increase in loading due to putrescible organic wastes must be considered, as well as direct interference with biological activity, aeration and sludge disposal. Wastes containing copper from copper-working mills, or arsenic and other metallic poisons from paint industries, have interfered seriously with the process, although there is a limited amount of tolerance to such substances, as there is to phenolic wastes. In concentrations of about 3 per cent or more, acids and alkalies are also detrimental to the activating organisms. Tar and mineral oil give rise to an important problem by preventing proper aeration and affecting the sludge.

Some industrial wastes, on the other hand, are believed to improve oxidation by conveying oxygen to the bacteria (Section 262) or otherwise acting as catalysts.

405. Partial Treatment.—An interesting characteristic of the activated-sludge process is its adaptability to partial treatment. If the removal of most of the colloids and bacteria and the

production of a moderate degree of stability are sufficient, the process can be stopped short of material oxidation of organic matter and nitrification. On the other hand, if complete stability and oxidation of a substantial quantity of organic matter are required, the process can be operated so as to accomplish this.

The process is furthermore adapted for use in connection with other oxidation devices such as trickling filters. It has been employed in this way at Birmingham, England, and Decatur, Illinois. The Hartley-type aeration tanks at Birmingham remove about 60 per cent of the fine suspended and colloidal matter

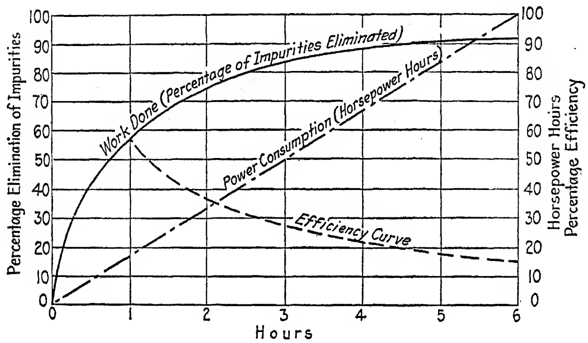


FIG. 210.—Elimination of impurities and power consumption during different intervals of time.

(80 p.p.m.) from settled sewage in about one hour's aeration. The effluent can be treated on the trickling filters at almost double the rate of dosing with unaerated sewage. Other advantages claimed for this combination of oxidation methods are the prevention of odors when the effluent is distributed over the filters, the elimination of the fly annoyance, and reduction of serious disturbances in the treatment process caused by variation in the character and quantity of industrial wastes.

Watson has shown (Fig. 210) that the *rate* of improvement secured at the first is greatly in excess of that secured in successive periods of the treatment, and has determined the relative efficiency with respect to power consumption under conditions applicable to Birmingham, England. According to Watson's studies, 56 per cent of the impurities were eliminated during the first hour, while this value was raised only to 74, 83 and 88 per cent during the second, third and fourth hours, respectively. An

example will illustrate the method of finding the efficiency curve. Calling the work done in the first hour 56 per cent with a power consumption of 16 hp.-hours, and the work done during the first 2 hours 74 per cent, requiring 33 hp.-hours, the relative efficiency for 2 hours' aeration is $\frac{74}{33} \times 16 = 36$ per cent.

During partial treatment a fully activated sludge is rarely produced and sludge conditioning must ordinarily be provided for the return sludge. The conditioning process is similar in character to reaeration. Simplex agitators in the sludge-conditioning tanks and air diffusers in the sludge-return channels accomplish this phase of the process at Birmingham.

406. Digestion of Excess Sludge.—Discussion of the treatment and disposal of activated sludge is deferred to Chapter XIX, excepting the question of digesting the sludge produced by this process. Where the high nitrogen content of activated sludge is not to be utilized in marketing the product as a fertilizer, digestion of the sludge alone or with fresh sewage solids produced by preliminary sedimentation of the sewage seems to present a satisfactory means of preparing the sludge for disposal. As removed from the final sedimentation tanks activated sludge has a high water content (97–99.5 per cent) and is extremely unstable, becoming septic in a short time. Digestion will increase the sludge density and anaerobic decomposition will produce a sludge which is similar in some respects to so-called Imhoff sludge.

Three means for digesting activated sludge suggest themselves, selection being determined largely by the method of pre-treatment in use:

1. Digestion with fresh solids in Imhoff tanks.
2. Digestion with fresh solids in separate sludge-digestion tanks.
3. Digestion alone or with screenings[1] in separate sludge-digestion tanks.

The factors governing the rate of digestion are the same for activated sludge as for fresh sewage solids, namely seeding, temperature and reaction, except that reaction is seemingly of less moment, because the pH value maintains itself at or above the neutral point. When activated sludge is digested together with fresh solids the ratio of activated to fresh solids enters into the problem. Tests by the Sanitary District of Chicago indicate

[1] Section 333.

that a lower total yield of gas results from mixtures containing more than 30 per cent by weight of volatile activated sludge (about 33 per cent of total solids on a dry basis). If we assume that 50 per cent of the solids in the raw sewage are removed by sedimentation, and that in the aeration tanks 90 per cent of the solids remaining after preliminary sedimentation are removed, the proportion of activated solids in the resulting mixture of activated and fresh solids will be $\frac{0.5 \times 0.9}{0.5 \times (1+0.9)} = 47$ per cent. According to Imhoff, the reduction in volume of activated sludge digesting with fresh solids is rapid, the water content decreasing by 7 or 8 per cent in 7 days, and the final sludge being as dense as ordinary Imhoff sludge (about 20 per cent solids). Since the volume of sludge to be digested is practically double that from preliminary sedimentation alone, digestion-tank capacities and sludge-disposal provisions must be doubled approximately. The gas yield will then be about twice as large. At Chicago the fresh solids and excess activated sludge from the North Side plant are to be digested in the Imhoff tanks of the West Side plant, the digestion-tank capacity being increased by 5 cu. ft. per capita above the 2 cu. ft. per capita for the West Side Imhoff-tank sludge. An allowance of 150 per cent rather than 100 per cent is thus made for excess activated sludge, tests at the Des Plaines plant of Chicago having shown that the digestion period of mixed activated and fresh solids is somewhat increased over that expected for fresh solids alone.

If excess activated sludge is to be introduced into the influent to two-story tanks, the bottom slopes of the flowing-through compartments should be steeper and the sludge slots wider than they would be where no activated sludge is introduced.

Rudolfs[1] found in a test at Milwaukee that at an average temperature of 81°F. digestion of activated sludge alone could be carried on with a digestion schedule of about 18 days, and that a well-draining, odorless 95-per cent sludge could be produced. Almost 50 per cent of the organic matter was decomposed, and 36 per cent of the grease. The nitrogen loss was 32 per cent. At Essen-Rellinghausen, Imhoff recorded a nitrogen content of 3.0 per cent for activated sludge and 2.5 per cent for fresh sewage solids. When the fresh solids were digested alone the nitrogen content was reduced to 1.2 per cent, whereas it maintained itself

[1] *Sewage Works Journal*, 1929; **1**, 146.

at 2.3 per cent for mixed solids. Rudolfs concluded from the Milwaukee tests that activated sludge digests more rapidly than fresh sewage solids and that no greater digestion capacity is required, notwithstanding the fact that more solids must be treated from the activated sludge process (without presedimentation) than from settling processes.

PRINCIPLES OF DESIGN

407. Factors Affecting Design.—There are certain general conditions which affect the design of activated-sludge units. These may be listed as follows:

1. Quantity and character of sewage to be treated, including the effects of preliminary treatment and the influence of industrial wastes.

2. Quality of effluent desired, including consideration of partial treatment by activated sludge and subsequent treatment by other oxidation devices.

3. Quality of sludge to be produced, including evaluation of its influence upon the treatment process itself and upon treatment, disposal and commercial utilization of the sludge.

These general factors influence in one way or another a number of design and operating factors, many of which are mutually dependent and must be so considered if the most effective and economical balance is to be obtained. Variation in one of these factors may be offset, within limits, by proper adjustment of the others. Chief among these factors which will be considered in subsequent sections are: (1) period of aeration; (2) quantity of air required, when diffused air is employed; and (3) proportion of sludge discharged into the sewage.

408. Period of Aeration.—A substantial length of time is required for the fine suspended and colloidal substances to become coagulated and for the absorption of colloidal matter by the return sludge and the newly formed floc.

The progressive nature of the changes which take place in the quality of the sewage during aeration is shown in Table 99, giving the results of applying a continuous supply of 160 cu.ft. of air per minute to sewage which originally had 235 p.p.m. of suspended matter. These tests were made in 1915 at the Milwaukee Testing Station by Copeland. The data in this table indicate that all that is necessary to obtain an effluent of the

desired quality is to select the proper detention period for the operating conditions. This, however, is not so simple as it appears, for these conditions, which are determined by factors such as the design of the tanks, the rate of application of the air, the character of the sewage, and the quality and volume of the return sludge, may vary greatly and continuously in the same plant.

TABLE 99.— RESULTS OBTAINED BY AERATION OF SEWAGE FOR VARIOUS PERIODS OF TIME[1]

Aeration, hours	0	1	2	3	4	5
Cu. ft. of air per minute	0	160.00	160.00	160.00	160.00	160.00
Cu. ft. of air per gallon	0	0.67	1.32	1.98	2.64	3.31
Appearance of settled liquor	Turbid	Clear	Clear	Clear	Clear	Clear
Stability, hours	0	2.00	33.00	120+	120+	120+
Bacteria removed, per cent	0	52.00	81+	92+	95+	98+
Free ammonia, p.p.m.	22.00	17.00	15.00	11.00	7.00	5.00
Nitrites, p.p.m.	0.08	0.00	0.95	1.75	2.20	2.50
Nitrates, p.p.m.	0.08	0.04	0.70	2.80	5.60	8.20
Dissolved oxygen, p.p.m.	0.00	0.30	1.90	4.30	5.90	6.70

[1] HATTON: *Eng. News*, 1916; **75**, 307.

The character of the sewage has a marked influence upon the required period of contact. A dilute fresh domestic sewage may become well coagulated in 3 to 4 hours, whereas a strong sewage containing industrial wastes may require 6 or even 8 hours, and it may be necessary to aerate for 8 or 10 hours, or even longer, industrial wastes high in colloidal matter, to secure the same relative degree of purification. For American municipal sewages the economic period of aeration appears to lie between 2 and 8 hours (see Tables 101 and 102). The nominal detention period in some of the bio-aeration tanks is longer than for diffused-air tanks. At Sheffield, England, it amounts to 15 hours and at Bury, England, where the Simplex system is employed, the tanks hold 10 hours' flow. At Essen-Rellinghausen, Germany, the units employing both diffused air and mechanical agitation hold 3½ hours' flow, the sewage being relatively weak. Tests with a Link-Belt agitator are said to indicate a required period of aeration of 6 hours. For partial treatment an aeration period of one or two hours has been employed, together with reaeration of the return sludge for several hours.

The period of aeration must naturally be longer when a highly nitrified effluent is to be produced than when clarification is the criterion of treatment.

409. Quantity of Air Required.—The quantity of oxygen necessary for the maintenance of aerobic conditions is relatively small. From a comparison of analyses of the air entering and the gases escaping from aeration tanks, Crawford and Bartow estimated that only about 5 per cent of the oxygen of the air introduced during the aeration process was utilized in maintaining aerobic conditions. In the case of strong tannery wastes the authors found that only 10 per cent of the oxygen in the air introduced disappeared during the process. As previously noted 30 per cent utilization of oxygen was obtained at Essen-Rellinghausen, Germany.

Experimenting to determine the actual quantity of oxygen necessary to stabilize sewage of "average strength," Clark found the average requirement to be 42.6 p.p.m., equivalent to 350 lb. of oxygen per 1,000,000 gallons. This is somewhat less than 1 per cent of the quantity which is used in the aeration process if the rate is 2 cu.ft. of air per gallon of sewage treated, or about 2 per cent in case 1 cu. ft. of air is sufficient for sewage of this character.

Where agitation is effected by blowing air into the sewage, the quantity of air required is very large. It is necessary to provide sufficient agitation to prevent the heavy suspended solids from settling on the air-diffuser plates and clogging them. As already pointed out, however, removal of suspended matter by fine screening may be a substantial aid in preventing such clogging. Even sewage which has passed a fine screen having openings ⅛ by 2 in. may contain sufficient solids of this character to cause clogging of the plates, if the air supply be permitted to fall much below 1 cu. ft. per minute per square foot of plate area. Finer screening might make it possible to reduce this quantity slightly. Sedimentation is more effective and will permit appreciable reduction in the quantity of air supplied (probably 15 to 20 per cent).

The volume of air required may be stated in two ways for purposes of comparison, (1) as cubic feet per gallon of sewage treated and (2) as cubic feet per square foot of tank surface per hour. Since the degree of compression varies for different plants, "free air conditions" are commonly specified, *i.e.* air at the prevailing atmospheric conditions of pressure, temperature and

moisture (commonly 30 in., 60°F. and 50 per cent saturation, respectively). Both methods of statement yield significant information as regards plant design and operation, the first being more widely used than the second. To change from cubic feet of air per gallon of sewage, to cubic feet of air per square foot of tank surface per hour, multiply 7.48 times the former value by the tank depth in feet and divide by the period of aeration in hours. Thus 1.0 cu.ft. per gallon in a 10-ft. tank with a period of aeration of 6 hours equals $\frac{1.0 \times 7.48 \times 10}{6} = 12.5$ cu. ft. per square foot per hour.

As a general statement, where the aeration is effected by blowing air into sewage passing continuously through tanks from 10 to 15 ft. deep, with diffuser-plate area equal to ⅙ the tank area, the quantity of air will probably not exceed the following (see also Tables 99, 101 and 102):

	Cu. ft. of free air per gallon of sewage
1. For rather weak domestic sewage	1.0
2. For strong municipal sewage containing some industrial wastes not particularly detrimental to bacterial life	1.5
3. For weak municipal sewage containing considerable industrial wastes, some of which are detrimental to bacterial life, as for example, acid-iron wastes	2.0
4. For strong municipal sewage containing considerable industrial wastes, some of which are detrimental to bacterial life, as for example, acid-iron wastes	4.0
5. For municipal sewage containing sufficient industrial wastes, not specifically inhibitive of bacterial life, to influence decidedly the composition of the sewage, *e.g.*, packinghouse, tannery, etc.	3.0
6. For strong packinghouse wastes	4.0
7. For strong tannery wastes containing a very large proportion of colloids, but settled before aeration	6.0

Where compressed air is used in connection with mechanical agitation the quantity required may be greatly reduced. At Essen-Rellinghausen a value of 0.13 cu. ft. per gallon was obtained. As the cost incident to the use of a large quantity of air is burdensome in operating expense and in the fixed charges (interest and depreciation) upon the installation, it is necessary to reduce it to the minimum practical limit.

410. Proportion of Return Sludge.—The proportion of return sludge at the influent end of the aeration units is commonly expressed as the percentage ratio of the volume of solids settling from a sample of mixed liquor in a given period of time. The volume is determined as indicated in Section 399, by use of a settling glass or centrifuge. Measurements also may be made on a gravimetric basis by finding the weight of the suspended solids. The return sludge commonly averages 2 per cent solid matter, and under these conditions the proportion of sludge fed into the inflowing sewage will generally correspond approximately to the ratio of sludge determined in a test sample, taken from the aeration tank and allowed to settle for 1 hour.[1]

In general, the quantity of 98-per cent sludge required for the treatment of moderately strong sewage will be equivalent to about 25 per cent of the volume of sewage treated (see Tables 101 and 102). This quantity may be stated also as about $\frac{25}{1.25} \times 0.02 = 0.4$ per cent of dry suspended solids by weight, referred to the total weight of mixed liquor.

Certain observers have found little difference in results obtained with sludge volumes varying from 7 to 30 per cent, but the consensus of opinion seems to be that there is an optimum ratio which depends upon the character of the sewage and sludge, the time of aeration and the volume of air supplied. With strong sewage, proportions as high as 50 per cent may be required, particularly when the sewage contains much colloidal matter. The volume of return sludge should naturally be kept as low as possible to secure economy of pumping and tank capacity, and because an unnecessarily large volume of sludge will require more air to keep it active. On the other hand the larger the proportion of sludge in the aeration tank the greater is its reserve capacity to deal with variations in the quantity and quality of the incoming sewage. The relation between proportion of return sludge and aeration period has been studied particularly by Harris and his collaborators at Glasgow, Scotland, working with a large-scale unit. Keeping the air supply and agitation constant, they determined the minimum sludge ratio and contact period to produce equal purification. The minimum effective proportion of sludge was found to be 8 per cent with a contact period of 4.2 hours. The product of these two values, 33.6, was called the *coefficient of interfacial contact*. The use of this coefficient is said to have been of practical value at Shieldhall (Glasgow) in

[1] The volume of return sludge should be recorded by a meter on the sludge conduit. The ratio of this volume to that of the sewage treated is an essential factor in design.

making it possible to maintain good operating conditions and increased plant capacity.

For partial treatment of sewage by the activated-sludge process, the volume of return sludge is usually small. At Birmingham, England, from 2 to 5 per cent was found to be sufficient in the partial treatment of settled sewage containing 80 p.p.m. of colloidal and finely divided solids.

411. Quantity of Sludge to be Handled.—The sludge deposited in the sedimentation units following the aeration tanks consists, broadly speaking, of the activated sludge which has been discharged into the influent augmented by the materials removed from the flowing sewage. Part of this sludge is again discharged into the influent of the aeration units as *return sludge*, part is disposed of as *excess sludge*. Both must usually be handled by pumps. The proportions, character and volumes of sludge normally handled are illustrated in the following schedule:

Volume of sewage treated	1,000,000 gal.
Water content of sludge	98 per cent
Specific gravity of sludge	1.005
Weight of solids produced (assuming 90-per cent removal of 300 p.p.m. of suspended solids, $0.9 \times 300 \times 8.33$)	2,250 lb.
Volume of sludge produced $\left(\dfrac{2{,}250}{8.33 \times 0.02 \times 1.005}\right)$	13,500 gal.
Volume of return sludge (assumed at 20 per cent of sewage volume)	200,000 gal.
Total sludge from sedimentation tanks	213,500 gal.
Return sludge $\left(\dfrac{200{,}000}{213{,}500}\right)$	93.7 per cent of total sludge volume.
Excess sludge $\left(\dfrac{13{,}500}{213{,}500}\right)$	6.3 per cent of total sludge volume; 1.35 per cent of sewage volume (values of 1.0 to 1.5 per cent are common).

As the density of the sludge may vary widely, the volumes handled may change considerably and approximately inversely as the solid content of the sludge.

412. Velocity of Circulation.—The velocity necessary to maintain circulation of sludge in the aeration units depends to a considerable extent upon the design of the tanks. It depends, too, upon the relative absence of heavy or gritty substances which

weight down the sludge or settle by themselves. If pocketing is prevented, the velocity of the main body of sewage may be made lower than when there is a chance for the establishment of eddy currents and dead areas. Velocities along the tank bottom are particularly important. Hurd found at Indianapolis that with spiral flow, minimum velocities of ½ ft. per second prevented deposition of solids. Design values vary from this up to about 1½ ft. per second, the figure used at Sheffield. The Sheffield value, however, represents the average channel velocity, and the bottom velocity is probably appreciably lower. Velocity of circulation must not be confused with the rate of longitudinal travel.

413. Arrangement of Aeration Units.—The general features of different aeration units have been described in Sections 392–394, and some of the design features are shown in Tables 101 and 102. As far as the arrangement of aeration units is concerned three types may be distinguished: (1) the once-through tank, (2) the endless channel and (3) the stage system. Aeration units employing diffused air generally conform to the first arrangement while mechanical agitation makes use of all three types.

In *once-through* tanks the sewage enters at one end and leaves at the end of a straight run or after passing around one or more longitudinal baffles or dividing walls. Displacement is more or less continuous from inlet to outlet, and short-circuiting is reduced (1) by inducing vertical and, in the case of spiral flow, transverse horizontal currents in the sewage, and (2) in some cases by transverse baffling. As in the case of sedimentation tanks, the flowing-through period may depart appreciably from the theoretical detention period (see Section 313).

In the *endless channel*, as developed at Sheffield, the sewage entering at one end of the aeration tank flows back and forth through 20 or more channels to the outlet end, where it is discharged over a weir into the settling chamber. From the outlet end of the aeration tank a channel passes back to the inlet end and thus makes it possible for a portion of the sewage to pass back and around the circuit again with the incoming sewage. The effluent contains a varying proportion of sewage which has passed only once through the series of channels, depending upon changes in the rate of inflow.

Hartley attempted to correct the deficiencies of the endless channel by introducing the *stage system* of tank arrangement. As used at Birmingham, the stage system may, broadly speaking,

be described as a "series arrangement" of endless channels, each stage consisting of a one-loop endless channel from which the sewage passes to a second, third, etc. The outlet from one loop channel to the next is located at a point which is as far from the inlet as possible. Interconnection of two or more loops has also been employed. Operation of Simplex aerators may be considered analogous to endless-channel operation when the units are arranged in parallel and to stage operation when they are arranged in series.

Arrangement of aeration units is predicated upon the choice of one of these systems, but also upon topographic conditions.

414. Tank Dimensions.—The capacity of aeration units is given by the detention period, allowance being made for the return sludge as well as the sewage. The choice of tank dimensions depends upon the system of treatment and upon local conditions. Deep tanks conserve area but require more power for compressing the air where diffusers are employed. Construction difficulties may also be encountered.

For use with diffusers and Simplex and Link-Belt agitators, deep tanks generally are favored. In diffused-air aeration greater depth provides a longer period of contact between air and sewage, and tanks have been made from 6 to 15 ft. deep (see Tables 101 and 102), the latter depth being common in this country. Bio-aeration units which depend largely on surface aeration, such as the Sheffield and Hartley tanks, are commonly quite shallow, being from 4 to 5 ft. deep.

Long tanks reduce short-circuiting, but the choice of length is also influenced by the desirable width. The latter is greatly affected by requisites of circulation and diffuser arrangement. Widths equal to the depth and up to over twice the depth have been employed. The ridge-and-furrow tanks at Milwaukee were made 22 ft. wide in order to secure an economical arrangement of the air piping and diffuser plates. The number of plates fed by a single line was restricted to nine 12-in. square plates, and an air main was installed along the center line of the tank, a narrow continuous channel being left in the ridges to permit draining the tank. The ridges have a slope slightly in excess of 45 deg.

Experiments at Indianapolis showed that spiral circulation could be maintained in a 15-ft. tank 30 ft. wide with an air flow of 0.6 cu. ft. per gallon of sewage. Circulation was aided by deflectors at the sewage surface. The Indianapolis tanks were

built 20 ft. wide and the deflectors were formed to give a slope of 6 vertical on 7 horizontal above the diffusers and 6 vertical on 5 horizontal on the opposite side. At Chicago they form an angle of 40 deg. with the horizontal. The corner between the tank wall and floor on the side opposite the diffusers is filled to aid circulation, the finished surface having a slope of 12 vertical on 7 horizontal at Indianapolis.

415. Air Diffusers.—Much study has been given to the subject of air diffusion, with a view to securing a diffuser that will meet the following requirements: (1) deliver the air to the sewage in a state of very fine division, (2) operate without becoming clogged, and (3) offer only a small frictional resistance to the passage of the air. A number of arrangements have been tried, such as perforated pipes, screen cloth, basswood and porous stone plates. The last have proved most satisfactory and economical. Two types are in common use in the United States, one manufactured by the General Filtration Company, Rochester, N. Y., and known as filtros plates, the other by the Norton Company, Worcester, Mass. Filtros plates are usually 12 in. square and about 1½ in. thick and are made of ground quartz sand and a siliceous binding material baked until hard. There are several grades of plates having the same degree of porosity but varying in frictional resistance. The grades commonly used in sewage treatment will pass from 10 to 15 cu. ft. of air per square foot per minute through a dry plate with a loss of head of 2 in. of water. The resistance of the plates is greater when they are wet and varies from 9 to 11 in. for a discharge of 2 cu. ft. per square foot per minute, to 10 to 12 and 14 to 16 in. when the air flow is 4 and 8 cu. ft. per square foot per minute, respectively. These differences are probably due to the drying-out of the plate with greater air flows.

Diffuser plates are commonly set in concrete holders, cast iron holders having caused trouble due to the clogging of plates with iron rust. Not more than 10 plates are set in a single holder.

The ratio of diffuser area to tank surface has generally been made somewhat greater for ridge-and-furrow construction than for spiral flow. For the former it appears to lie between 1 to 4 and 1 to 7, although smaller ratios have been adopted in some cases. For the latter, ratios between 1 to 9 and 1 to 15, or even smaller, have been employed (see Tables 101 and 102).

Diffuser plates become clogged if the compressed air contains oil and dust. For this reason, it is important to provide clean air and to have the air compressed by a mechanism that does not allow oil to enter the air, even in minute quantities.

The air is best supplied to the diffusers through pipes protected against rust, for if rust forms in these pipes it is likely to clog the pores of the diffusers.

416. Air Piping.—In designing the system of air piping, economy of materials must be balanced against power consumption. Fuller and McClintock[1] state that air velocities should generally not exceed 2,000 to 3,000 ft. per minute, and that the distribution loss, including the plate loss, should usually be below 1.0 to 1.5 lb. per square inch. To secure equality of air distribution the friction loss in the mains must be kept low or separate regulating valves must be provided for each diffuser unit. The latter seems to be the preferred practice in England. At Milwaukee one valve was provided to control 1,000 plates. Piping should be accessible, corrosion resistant, drainable and provided with traps to collect condensed moisture.

Estimates of pipe losses can be made by a number of formulas. The Sanitary District of Chicago has tested and employed the Fritzsche formula which may be stated as follows:

$$s = \frac{1.268Q^{1.852}t}{1{,}000pd^{4.973}}$$

where s = drop in pressure per 1,000 feet of pipe in lb. per sq. in.
Q = cu. ft. of free air per minute at 60°F.
t = absolute temperature in °F. = recorded temperature in °F. +459.6
p = absolute pressure in lb. per square inch = gage pressure +14.7
d = diameter of pipe in in.

Special allowances must be made for losses due to bends, entrance, venturi meters, valves and the like.

417. Types of Air Compressors.—A number of types of air compressors have been used in activated-sludge plants, among which the following may be mentioned: (1) single-stage piston compressors; (2) positive pressure blowers; (3) centrifugal compressors; and (4) Nash Hytor compressors. The last three mentioned are illustrated in Fig. 211. The first one has not been used much in this country, as it is not well suited to the low pressures

[1] "Solving Sewage Problems," pp. 446 and 447.

employed and because it is difficult to keep cylinder oil out of the compressed air.

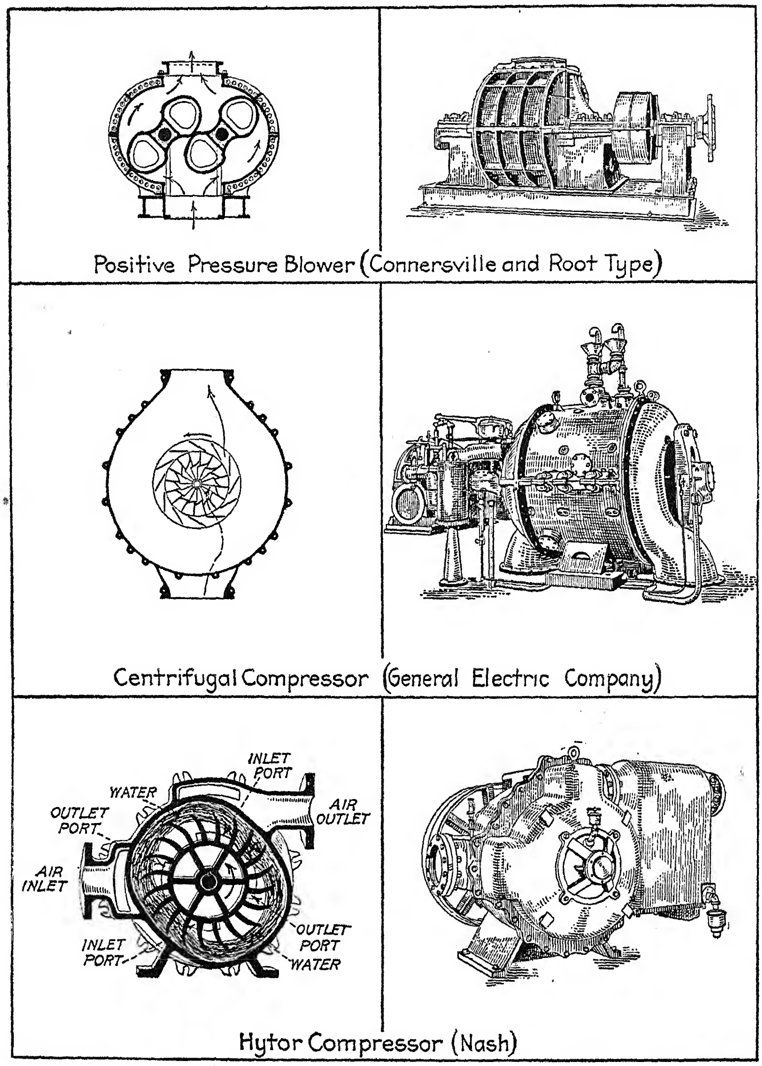


FIG. 211.—Types of air compressors.

In selecting the type of blower, there should be borne in mind the probability that impurities in the air will gradually accumulate on the surface or in the pores of plate diffusers, resulting in

partial clogging and a gradual increase in the frictional resistance. The initial operating pressure may have to be increased considerably from time to time on this account, and the blower should be selected with this in view. It is also important to provide a flexible blower plant capable of adjustment in air supply to the volume and quality of sewage and other conditions subject to frequent change. Variation in volume of air can be secured by throwing in or out of service one or more compressors. Greater flexibility, however, can be secured by the use of centrifugal compressors (Fig. 211) with which the volume of air furnished can be varied by opening or closing the blast gate in the suction pipe. The power required to drive this type of compressor varies approximately with the volume of air used, when operating at a constant speed. The possibility of varying the volume of air compressed, without wasting power, is one of the advantages of this type of machine. On the other hand, this type of compressor furnishes air at a uniform pressure, which can only be changed by changing the speed of the impeller, which with certain types of driving apparatus is not practicable. In such cases, the pressure must be high enough to meet all emergencies. Therefore at all other times there is a waste of power in compressing the air to a higher pressure than is necessary.

At a given speed a practically constant volume of air is furnished by positive pressure blowers and Hytor-type compressors alike (Fig. 211), the pressure depending upon the resistance offered, as by the head of water against which the air is introduced into the aeration tanks, and by the friction in pipe lines and diffuser plates. One of the advantages of these types of blower is that they will build up pressure sufficient to overcome increased frictional resistance in the filtros plates and that it is not necessary to waste power by raising the pressure unnecessarily high, as in the case of the centrifugal blower. On the other hand, if great flexibility be required in the volume of air, there may be a waste of power due to compressing an unnecessarily large volume. Probably in most activated-sludge plants sufficient flexibility can be obtained by providing a proper number of units of appropriate size and varying the number of units in use.

418. Power Required for Compressing Air.—The pressure to which the air must be raised is equal to the pressure due to the head of sewage in the tanks plus distribution losses in piping and plates, allowing for a reasonable factor of safety.

Since heat is generated in compressing air, compression is not *isothermal* but *adiabatic*, the heat not carried off swelling the volume of the air and increasing the power expended by an amount corresponding to the increase in volume compressed.

The amount of power required and its cost can be estimated from data in Table 100, by applying the assumed efficiency and other data peculiar to the case under consideration. The efficiency which may be expected from a high-grade electric-motor-driven blower plant will vary according to the type and size of machine, but in general may be assumed to be from 60 to 70 per cent.

TABLE 100.—POWER REQUIRED FOR COMPRESSING AIR

Final pressure of air (lb. per sq. in.)	Theoretical work to compress 1 mil. cu. ft. of free air (horsepower-hours)	Theoretical power to compress 100 cu. ft. free air per minute (horsepower)	Theoretical cost of compressing 1 mil. cu. ft. free air with elec. power at 1 ct. per kw.-hr. (dollars)
1	72.3	0.43	0.539
2	144.0	0.86	1.074
3	200.8	1.20	1.498
4	265.2	1.59	1.978
5	325.8	1.95	2.430
6	384.7	2.31	2.869
7	442.5	2.66	3.301
8	490.2	2.94	3.656
9	543.4	3.26	4.053
10	596.5	3.58	4.449
12	697.1	4.18	5.199
14	785.4	4.71	5.858
16	875.9	5.26	6.533

NOTE.—This table is based on the assumption that the air is compressed under adiabatic conditions (without cooling) as is the practice in nearly all blower work, from atmospheric pressure (14.7 lb. per square inch) and an initial temperature of 60°F. Weight of 1 cu. ft. free air = 0.0764 lb. Slightly lower power consumption may be obtained with good water-jacketed reciprocating compressors.

A convenient approximate unit of power to bear in mind is that 30 hp. per hour will be required for furnishing 1.5 cu. ft. of free air per gallon of sewage for the treatment of 1,000,000 gal. of ordinary municipal sewage in 24 hr., in tanks 10 ft. deep.

419. Cleaning Air.—To prevent the clogging of plate diffusers the air should be thoroughly cleaned before being compressed.

The means for accomplishing this are generally the same as those employed in ventilation of buildings and factories. They may be classified as (1) cloth filters, (2) air washers, and (3) oil cleaners of the Midwest type. For cloth filters canton flannel and 10 or 12-oz. duck are employed. The permissible rate of filtration is about 4 cu. ft. of air per square foot of cloth per minute. In air washers, jets of water are sprayed through the air and carry down the dust and other floating matter. Oil cleaners consist of cells filled with a metal medium coated with a light oil. The air passages are tortuous and the impurities are removed on the contact surfaces.

Milwaukee employed oil cleaners, after experiencing troubles with air washers due to freezing. Air washers are used at Indianapolis and at the North Side plant of Chicago. The latter plant uses in addition oil cleaners and heaters. Cloth filters are employed at the Des Plaines plant of Chicago.

420. Mechanical Agitators.—The design of agitators in bio-aeration units is commonly in the hands of the proprietors of the design patents. The power required to drive the agitators is said to be approximately the same as for compressing the air in diffused-air plants. At Sheffield the horsepower per million gallons is 29, at Bury (Simplex system) it is 20. By combining the use of diffusers and mechanical agitation Imhoff has reduced the power to 7 hp. per million gallons. The character of sewage treated must naturally be considered to appreciate the significance of these figures.

It is essential that moving parts be of sturdy construction, and that bearings and other parts liable to wear be accessible. Units should be so arranged that failure of one mechanism will not jeopardize the operation of the entire plant.

421. Final Settling Tanks.—The design of settling tanks in which the activated sludge is removed from the aeration-tank effluent does not differ materially from the design of plain sedimentation tanks. The flocculent nature of the sludge must, however, be taken into account. Another factor to be borne in mind is that the tanks must provide temporary storage of sludge so that a sufficient amount of activated sludge will be available to meet fluctuations in the quantity of incoming sewage. The variations to be allowed for are differences in day and night flow, week-day and holiday flow, and dry-weather and storm flow. Variations in sludge density must be properly allowed for in

providing storage. Since activated sludge decomposes rapidly, however, excessive detention should not be permitted.

Both horizontal and vertical flow are employed (see Tables 101 and 102). Copeland found that the horizontal velocity must not exceed 3 ft. per minute and it is customary to keep the vertical velocity below 30 ft. per hour. Detention periods of 1 to 6 hours are used, 2 hours being a common allowance.

For vertical flow, tanks of the Dortmund type are generally employed. Bottom slopes in excess of two vertical on one horizontal are needed to permit the flocculent sludge to slide to the bottom. To reduce the tank depth several hoppers are commonly provided for large tanks.

Horizontal-flow tanks are generally equipped with mechanical sludge removal devices, such as the Dorr and Link-Belt mechanisms. In dealing with activated sludge, the Link-Belt mechanism is said to operate best when the flights move sludge in the same direction as the flow. A new device for sludge removal has been developed at Milwaukee. It is known as the Tow-Bro sludge remover and consists of radial arms similar to the Dorr mechanism, but provided with pipes with multiple inlets instead of plows. The sludge passes into the pipes and is removed as rapidly from the periphery of the tank as from the center.

As the area of the sedimentation tank with respect to the volume of mixed liquor which it can successfully handle is an important consideration in dealing with flocculent activated sludge, it has been found convenient to use the unit of horizontal area in computations relating to sedimentation tanks. As a result of experiments at Milwaukee, the sedimentation tanks were designed to provide 1 sq. ft. of tank area for each 1,600 gal. of mixed liquor to be treated daily, measured at the time of assumed maximum flow of sewage, such rate being expected to occur only in times of storm. This is equivalent to 1 sq. ft. of tank surface for each 850 gal. of sewage, based upon the average daily flow.

The extent of treatment in the aeration tanks has a very important effect upon the process of sedimentation. It has been proved in some cases that aeration sufficient to produce a satisfactory effluent, if the floc could be removed, was insufficient to produce a floc which would readily settle in sedimentation tanks, or a reasonably dense sludge. In these cases, therefore, the extent of aeration may be controlled by the requirements of

the sedimentation process or by the required density of the sludge.

Aeration tanks of the Simplex type may include sedimentation compartments. These are formed by partitions in the corners of square tanks. Sewage overflows into them or is directed into them by the blades of the agitator. Part of the sludge slides through an opening back into the aeration compartment proper.

422. Piping and Pumps for Return Sludge.—Sludge from the sedimentation tanks contains so much water that it will flow readily in pipes. In fact in computing the size of pipes and conduits to transport sludge, coefficients of discharge but slightly lower than for water are used. Where sludge is aerated during its flow through conduits, an additional allowance may be advisable on account of interference with flow caused by cross-currents. In some computations the authors have used a coefficient as low as $C = 80$ in the Hazen and Williams formula, to allow for both of these influences. This value is in agreement with the suggestion of Clifford[1] that for practical purposes and until more accurate data are available the pressure drop for a 90-per cent sludge may be taken as approximately $1\frac{1}{4}$ times that of water.

The velocity in sludge pipes and conduits should be not less than 2 ft. per second, if deposition of solids is to be avoided, which is essential. Where the velocity must be less than this, air should be utilized to secure agitation and to maintain sludge activity.

Activated sludge may be raised by centrifugal and other types of pumps, by air lifts, and by compressed air ejectors. It is important to avoid violent agitation which will break up the floc and reconvert it into a colloidal state. There is no serious danger of this in slow or moderate speed centrifugal pumps of suitable design, and even less in air lifts and ejectors. Where the last-named are used, the ejector pot should be so designed as to prevent forming a layer of sludge on the bottom of the pot.

423. Examples of Aeration-tank Design.—The design characteristics of a number of activated-sludge units are shown in Tables 101 and 102. In making comparisons the character of the sewage should always be taken into consideration. This is particularly true in comparing European and American designs. Furthermore, it must be remembered that in a number of European

[1] Conference on Sanitary Engineering, 1924.

TABLE 101.—DESIGN CHARACTERISTICS OF TYPICAL ACTIVATED-SLUDGE PLANTS EMPLOYING DIFFUSED AIR

	Milwaukee, Wis.	Indianapolis, Ind.	Chicago, Ill. (North Side)	Decatur, Ill.	Manchester, Eng. (Withington)	Houston, Tex. (North Side)	Pasadena, Cal.	Toronto, Can. (No. Toronto)
Date of construction	1925	1924	1926	1927	1923	1917	1924	1929
Population served	575,000	360,000	800,000	50,000	18,000	67,000	120,000	50,000
Preliminary treatment								
Grit chambers	Yes	Yes	Yes	Yes	None	Yes	None	Yes
Fine screens	Yes	Yes	None	None	None	None	Yes	None
Sedimentation, hours	None	None	0.5	1.1	0.7	None	None	1.6
Aeration tanks								
Type	Ridge and furrow	Spiral circulation	Spiral circulation	Spiral circulation	Spiral circulation	Ridge and furrow	Ridge and furrow	Spiral circulation
Number	24	6	36	6	2	4	30	4
Length, ft	236	238	420	120	177	280	67.5	163
Width, one channel, ft	22	20	34.8	16.	6.2	9	10	13
Effective depth, ft	15	15	15	14.5	7	9.8	15	10.5
Number of passes per tank	2	4	1	1	3	2	1	2
Period of aeration, hours	6	4.7	6	2.5	7	2.3	4	4
Air consumption, cu. ft. per gallon	1.5	1	0.8	1	0.9	1.4	1	1
Return sludge, per cent of sewage flow	25	20	25	10	15	25	33	25
Ratio area of diffusers to area of tank	1:4	1:13	1:9	1:9	1:17	1:7	1:6	1:10
Final sedimentation tanks								
Type	Dorr	Hopper-bottom	Dorr	Dorr	Hopper-bottom	Hopper-bottom	Dorr and Hardinge	Fidler
Number	11	12	30	2	1	40	5	2
Inside dimensions, ft	98 (diameter)	78 × 42	77 × 77	77.5×77.5	36 × 36	19 × 10	50 × 50	65 × 65
Effective depth, ft	15	15	16	13.7	21	22	16	14.8
Detention period, hours	1.7	1.6	2	2.6	2	1.5	2	2
Flow of effluent, gal. per square foot per day	850	1,380	980	830	1,000	1,330	880	710
Reaeration tanks								
Number	None	None	None	None	1	4	5	None
Inside dimensions, ft	....	....	....	....	160 × 6	280 × 9	67.5 × 10	
Effective depth, ft	....	....	....	....	5.5	9.8	15	
Detention period, hr	....	....	....	....	5	2.3	4.5	
Air consumption, cu. ft. per gallon of sewage	....	....	....	....	....	0.7	0.6	

TABLE 102.—DESIGN CHARACTERISTICS OF TYPICAL ACTIVATED-SLUDGE PLANTS EMPLOYING MECHANICAL AGITATION

	Sheffield, Eng.	Bury, Eng.	Birmingham, Eng.	Essen-Rellinghausen, Germany	Princeton, Ill.
Date of construction	1927	1925	1927	1925	1927
Population served	530,000	29,000	260,000	37,000	3,500
Preliminary treatment					
Grit chambers	Yes	None	Yes	Yes	Yes
Fine screens	None	None	None	None	None
Sedimentation, hours	8	6	10	0.3	1
Aeration tanks					
Type	Sheffield	Simplex	Hartley	Paddles and compressed air	Simplex
Number	12	3	1	4	5
Length, ft	250	171	180	....	22.2
Width, one channel, ft	6	21	4.7	10	22
Effective depth, ft	4.4	9	4	10	12
Number of passes per tank	21	1	19	2	1
Period of aeration, hours	15	10	1	3.5	8
Air consumption, cu. ft. per gallon	....	....	....	0.13	
Return sludge, per cent of sewage flow	15–20	10.5	2–5	8	18
Final sedimentation tanks					
Type	Dortmund	....	Radial-flow	Dortmund	Link-Belt
Number	108	9	3	2	2
Inside dimensions, ft	25 × 25	21 × 21	60 × 60	66 × 33	48 × 9.5
Effective depth, ft	22.5	20	32	25	11.8
Detention period, hours	6.5	6	2	1	2
Flow of effluent, gal. per square foot per day	267	302	833	1,330	470
Reaeration tanks					
Number	None	None	15	None	None
Inside dimensions, ft	....	....	25 × 25		
Effective depth, ft	....	....	10		
Detention period, hours	....	....	18		

plants the activated-sludge units were built into existing tanks used previously for other types of treatment.

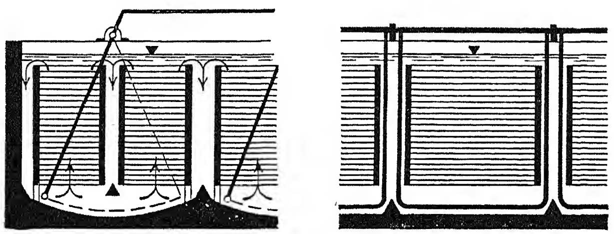
CONTACT AERATORS

424. Operating Factors.—Contact aerators (Fig. 212), first suggested by Buswell, occupy a position more or less intermediate between sewage filters, especially trickling filters, and activated-sludge tanks. Imhoff[1] classifies them as contact beds which operate while continuously submerged in sewage, aerobic conditions generally being maintained by blowing air through the contact material. The air, when used, performs the further function of maintaining circulation in the units. As developed in Germany, complete treatment is secured by arranging the units in two or three stages, the first stage alone being employed when only partial treatment is to be secured and further oxidation is to take place in activated-sludge tanks or trickling filters. The three stages may be compared with the three zones of self-purification observed in polluted streams—polysaprobic, mesosaprobic and oligosaprobic. This comparison is based upon the nature of the organisms developing upon the contact material and the changes brought about in the sewage treated. Differentiation from other treatment methods is particularly marked in the first stage in which sewage fungi dominate, and the high-molecular, organic dissolved and colloidal substances are converted into simpler organic substances but not into mineral matter. These intermediate products of decomposition are thrown out of solution by the life processes of the organisms, and to a large extent are changed into living cell matter. Growths must, therefore, be removed more or less continuously, together with such suspended matter as settles on the contact surfaces. Removal is accomplished by agitation or, more frequently, by increased use of air. The latter may be secured by shutting off the air from all but one unit at a time, say once a day in the evening. Since aerators seem best fitted for the removal of dissolved and colloidal substances, they commonly are made to follow presedimentation tanks. The growths and solids flushed from them are caught in final settling basins. As shown in Fig. 212, presedimentation and final clarification may be secured by building the units into the central flowing-through compartment of two-story tanks. Separate sedimentation tanks

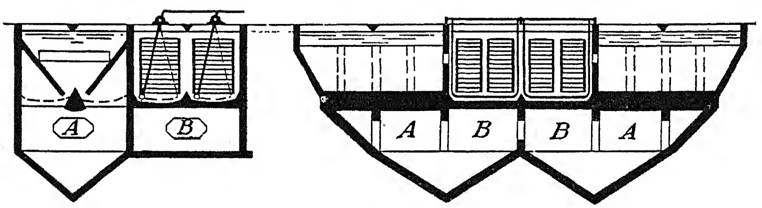
[1] Imhoff-Fair: "The Arithmetic of Sewage Treatment Works," p. 61.

must otherwise be provided. Where more than one stage is employed settling units must be interposed between the successive aerators.

Circulation of the sewage through the contact material is important and is obtained by confining the contact material in boxes open at top and bottom which do not quite take up the width of the tank. Air, blown into the boxes from below, drives the sewage through them and sets up a return current in the space between the boxes and the sides of the tanks. The sewage circulates not once but many times.



a.—Built into single-story tank.



b.—Built into two-story tank.

FIG. 212.—Contact aerators.

The final stage of treatment commonly is accomplished in ½ to 1 hour, while complete treatment requires about 4 hours.

425. Operating Results.—Tests of contact aerators, operated with presedimentation and final clarification, as employed in Germany for partial treatment of sewage (first stage treatment), are reported by Mahr and Sierp.[1] Table 103 gives some of the German findings together with a statement of certain design features. The data indicate reductions in organic matter similar to those obtained in the United States by sedimentation alone. Overloading, while producing a poorer effluent, it is stated, does not otherwise affect the working of the process. This is claimed as a distinct advantage over other biological treatment methods.

[1] *Technisches Gemeindeblatt*, 1928; **31**, Nos. 10 and 11.

Contact aerators, furthermore, are said to be relatively tolerant to toxic substances and have been employed successfully in treating phenolic wastes and wool-scouring wastes.

TABLE 103.—DESIGN AND OPERATING CHARACTERISTICS OF CERTAIN GERMAN CONTACT AERATORS

	Hagen	Velbert	Blanken-stein	Menden
Rate of sewage flow (day flow) gal. per day	683,000	1,140,000	160,000	1,140,000
Character of sewage	Domestic	Domestic	Septic, strong	Strong, domestic
Contact units				
Surface area, sq. ft	264	700	240	515
Effective depth, ft	2.5	4.8	4.9	8.5
Capacity, cu. ft	650	3,400	1,180	4,400
Contact period, min	10	32	80	40
Air supply				
Cu. ft. per gallon	0.44	0.19	0.28	0.33
Cu. ft. per square foot per minute	0.8	0.2	0.13	0.5
Method of distribution	Swinging pipe	Swinging pipe	Pipe grid	Swinging pipe
Average operating results, p.p.m.				
Oxygen consumed, influent	244	269	302	341
effluent	184	163	194	141
per cent reduction	25	39	36	59
B.O.D., influent	134	219	362	240
effluent	97	139	228	74
per cent reduction	28	37	37	69
Ammonia N., influent	33	44	53	45
effluent	29	33	49	28
Organic N., influent	21	28	17	25
effluent	11	23	11	7

426. Design Factors.—Some of the design characteristics of German contact aerators are shown in Table 103. German experience with one-stage units, as given by Mahr and Sierp[1] and Imhoff[1] may be summarized as follows:

Pre-sedimentation period	¾ to 1 hour
Dimensions of contact units	
Width of boxes	Less than 5 ft.
Effective depth of boxes	2 to 10 ft.
Length of boxes	About 10 ft.
Capacity of all boxes in terms of detention period	10 to 80 min.
Depth of top of box below sewage level	2 to 3 in.
Width of space between boxes and between walls and boxes	0.5 to 1 ft.

[1] Loc. cit.

Air supply	
Per gallon of sewage	0.13 to 0.4 cu. ft.
Per square foot of horizontal box area per minute	0.1 to 0.8 cu. ft.
Final sedimentation period	½ to 1 hour
Required increase in sludge-digestion capacity	10 per cent

Many varieties of contact material have been employed by Buswell and German workers. The most satisfactory ones for first-stage treatment seem to be (1) laths, or stringers, about 1 by 2 in. in cross-section, and (2) mats of veneer or basket wood. Coke, cinders and brushwood have not proved satisfactory as first-stage contact material, chiefly due to clogging. Where complete treatment is used it seems safe to construct the third-stage unit with coke or similar material. The laths are laid parallel to the air pipes, in a horizontal position but tilted, edge upward, so as to reduce the surface upon which sludge may settle. The mats are placed in a vertical position, parallel to the air pipes, and ¾ to 1½ in. apart. Air distribution by a perforated pipe swinging from side to side like a pendulum is held to be most satisfactory. A stationary pipe grid may be substituted, especially when industrial wastes, such as wool-scouring wastes, require large volumes of air. The pipes are generally 2 in. in diameter with perforations on the bottom side ⅛ to ¼ in. in diameter and spaced 6 in. apart, giving a combined area 20 to 50 per cent greater than the cross-section of the pipe. The pipes swing back and forth from two to six times in a minute. The swinging pipes are attached to the air mains by short sections of rubber hose, a valve being provided to control the air flow. The tank bottom is curved to fit the arc of the swinging pipe, a clearance of 2 in. being allowed. Sludge deposits are flushed out by the air currents. This is essential.

Box widths are limited to ensure continuous upward movement of the sewage. Depths are made as great as possible to economize air, the supply per square foot being, if anything, more important than the supply per gallon, since circulation is an all-important factor.

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Problems

1. If the mixed liquor at the influent end of an aeration tank contains 0.2 per cent suspended matter by weight, what per cent by volume of 97.5-per cent activated sludge has been discharged into the influent?

2. An aeration period of 6 hours is to be provided.

(*a*) What must be the capacity of an aeration tank treating 1 m.g.d. of sewage with 20 per cent return sludge on a 98-per cent sludge basis?

(*b*) If the moisture content of the sludge is increased to 99 per cent, what should be the percentage of return sludge and the tank capacity?

Ans. (*a*) 300,000 gal.
(*b*) 40 per cent; 350,000 gal.

3. The fresh solids and excess activated sludge from a 1-m.g.d. plant using 1 hour presedimentation are to be digested in separate sludge-digestion tanks. Assume that the sewage contains 300 p.p.m. of suspended solids. Estimate (*a*) gallons of fresh sludge per day; (*b*) gallons of excess activated sludge per day; (*c*) ratio of activated sludge to fresh sludge (by weight); (*d*) capacity of digestion tanks.

4. Excess activated sludge from a 1-m.g.d. plant employing fine screens is to be digested. Estimate the capacity of sludge digestion tanks, assuming that the sewage contains 300 p.p.m. of suspended solids.

5. How many cu. ft. of free air are commonly required to treat 1 m.g.d. of sewage? With spiral-flow tanks 15 ft. deep and 20 ft. wide, what will usually be the quantity of air per square foot of tank area per hour, and what the amount of air per square foot of diffuser plate?

6. On the basis of the Glasgow findings what must be the per cent of return sludge in the tanks for a 6-hour aeration period?

Ans. 5.6 per cent.

7. Choose the dimensions of (*a*) a spiral-flow tank, (*b*) a ridge-and-furrow tank, treating 1 m.g.d. of sewage. Include the diffusers.

8. Estimate the pressure to which air must be raised to supply 120 cu. ft. of free air per minute at 60°F. through 500 ft. of 3-in. pipe to diffuser plates passing 4 cu. ft. per square foot per minute situated in a tank 15 ft. deep.

Ans. 7.4 lb. per square inch.

9. Estimate the power required for compressing 12,000 cu. ft. of air per minute to 8 lb. per square inch. Assume that the air is at atmospheric pressure and a temperature of 60°F. before compressing. *Ans.* 353 hp.

10. How many square feet of cloth filter are required to clean 5,000 cu. ft. per minute of air? *Ans.* 1,250 sq. ft.

11. Choose the dimensions of a Dortmund-type final settling tank receiving the effluent from an aeration unit treating 1 m.g.d. of raw sewage.

12. Choose the tank and box dimensions of a contact aerator (first stage) treating 1 m.g d. of settled sewage.

CHAPTER XIX

TREATMENT AND DISPOSAL OF SEWAGE SLUDGE

427. The Problem of Sludge Disposal.—The solids removed in one way or another from the liquid sewage in sewage treatment works constitute the most important by-product of the treatment processes. These solids include grit, screenings and sludge, the last being by far the largest in volume. The disposal of grit and screenings has been taken up in Chapter XV, and it has been pointed out that screenings are sometimes disposed of together with sludge.

The daily volume of sewage handled in treatment works of other than rural communities commonly runs into millions of gallons and the volume of sludge produced, therefore, generally runs into thousands of gallons. The problem of dealing with such quantities of material is naturally one of considerable magnitude, especially when it is remembered: (1) that the sludge is made up in considerable proportion of substances which are responsible for the offensive character of untreated sewage; (2) that, except in the case of well-digested sludge, these substances are still capable of rapid decomposition in the state in which they are produced by the sewage treatment process; and (3) that only a small part of the sludge is solid matter. How to prepare the sludge for disposal without creating offense, how to get rid of the treated or untreated sludge as economically as possible, and how to secure its utilization as a fertilizer, where circumstances justify, are matters to be considered in this chapter.

428. Volume of Sludge Produced by Different Sewage-treatment Processes.—The quantities and characteristics of sludge produced by different processes of sewage treatment have been discussed in preceding chapters together with a consideration of the factors influencing its production. For purposes of comparison and reference a summary of typical yields is given in Table 104.

Appreciable departures from these values are to be expected. From what has been said in connection with the treatment

processes producing the different sludges, the student should be able to explain the relative variations in sludge volumes, water content, weight of dry solids and specific gravity of wet and dry sludges. Several ways of expressing the quantities of sludge are given in the table. Gallons per million gallons of sewage treated is a unit in common use by plant operators. Engineers use in addition cu. ft., cu. yd. and lb. per million gallons; cu. ft., cu. yd. and lb. per capita or per 1,000 population daily; and tons per capita or per 1,000 population yearly. Sludge volumes are compared also on the basis of 90-per cent water content.

Table 104.—Nominal Quantities and Characteristics of Sludge Produced by Different Treatment Processes

	Treatment Process					
	Activated sludge	Chemical precipitation	Plain sedimentation	Single-story septic tanks	Imhoff or separate digestion tanks	Trickling-filter humus tanks
Normal volume of sludge, gal. per million gallons of sewage	13,500	5,000	2,500	900	500	500
Cu. ft. per 1,000 population daily	181	67	33	12	6.7	6.7
Moisture, per cent	98	92.5	95	90	85	92.5
Specific gravity	1.005	1.070	1.020	1.040	1.040	1.025
Dry solids, lb. per million gallons of sewage	2,250	3,300	1,080	810	690	320
Lb. per 1,000 population daily	225	330	108	81	69	32
Approximate volume on basis of 90-per cent sludge, gal. per million gallons of sewage	2,700	3,750	1,250	900	750	375
Specific gravity of solid matter	1.25	1.93	1.40	1.40	1.27	1.33

As the moisture content of sludge is reduced it changes its state from that of *watery sludge*, first to *sludge paste*, next to *sludge cake* and finally to *sludge grains*. Watery sludge contains so much water that the sludge flows by gravity, and is pumped readily, cohesion of the particles being slight. The point at which the sludge will no longer flow varies for different sludges. Imhoff sludge, for example, will flow when it contains 80 per cent moisture, while activated sludge of this water content is distinctly a paste. This difference may be explained in part by the fact that gas bubbles are entrained in Imhoff sludge and increase its fluidity. When the water content of sludge is reduced to 50 or 60 per cent, it can be handled readily by a shovel or fork and

is called *spadable*. Sludge cake contains less water than is required to fill the natural pore space between the particles. The volume then no longer depends upon the water content. The sludge finally becomes granular when its moisture is reduced below 10 per cent, a requirement which generally must be met when the sludge is to be marketed as a commercial fertilizer.

429. Character of Sludge Produced by Different Sewage-treatment Processes.—From the standpoint of sludge treatment and disposal, factors other than volume and water content enter into the problem. Among them may be the appearance and putrescibility of the sludge, its digestibility, dewatering characteristics and fertilizing value. Appearance (unsightliness), putrescibility (odor production) and dewatering characteristics (particularly air drying) must usually be considered when the sludge is to be disposed of without treatment other than perhaps drainage to a spadable condition by running it onto beds of sand or other porous material. Digestibility is of importance if the sludge, excepting Imhoff sludge (which is already well digested) and some septic-tank sludges, is to be digested alone or mixed with other sludges in two-story or separate digestion tanks. Dewatering characteristics, with special reference to centrifuging, filtration and heat drying, generally become of moment when the moisture content is to be reduced by mechanical means, particularly in connection with the preparation of the sludge for commercial utilization of its fertilizing constituents.

The character of different sludges as far as it affects appearance, putrescibility, digestibility and air drying, is discussed below. Their properties as regards centrifuging, mechanical drying and use as fertilizers will be considered in subsequent sections of this chapter.

Activated sludge generally has a brown flocculent appearance. If the color is very dark the sludge may be approaching a septic condition. If the color is very light there may have been under-aeration with a tendency for the solids to settle slowly. The sludge, when in good condition, has an inoffensive earthy odor, but sometimes has a tendency to become septic rather rapidly and then has a disagreeable odor of putrefaction. It will digest readily alone or mixed with fresh sewage solids. If the sludge is flowed over a sand bed to a depth greater than 4 or 5 in., the suspended matter settles rapidly and clogs the pores of the sand before the water can pass through them, thus making the drying process a slow one even under the most favorable conditions, because a large part of the water can escape only by evaporation.

Sludge from chemical precipitation basins is usually black in color, though its surface may be red if it contains much iron. The odors from it may be objectionable, but not so bad as those from plain sedimentation sludge. While it is somewhat slimy, the hydrate of iron or aluminum in it makes it gelatinous. If it is left in the tank it undergoes decomposition like the sludge from plain sedimentation, but at a slower rate. It gives off gas in substantial quantities, and its density is increased by standing. When spread on drying beds the water will gradually drain away or evaporate, leaving after several weeks a stratum of slimy sludge of about the consistency of lard, containing 70 to 80 per cent of water.

Sludge from plain sedimentation basins is gray in color, slimy in consistency, and in most cases possesses a very offensive odor. It will digest readily under proper conditions of operation, and can be dried by spreading it on porous beds, but it must be spread thinly in order to enable the water to percolate away rapidly.

Sludge from single-story septic tanks is black and, unless well digested by long storage, is offensive on account of the hydrogen sulphide and other gases it gives off. The sludge can be dried on porous beds, if spread out in thin layers, but objectionable odors are to be expected while it is draining, except when the sludge is well digested.

Imhoff sludge contains an exceptionally large quantity of gas. When thoroughly digested it is not offensive, its odor being relatively faint and like that of hot tar, burnt rubber or sealing wax. When drawn off on porous beds in layers 6 to 10 in. deep, the solids first are carried to the surface by the entrained gases, leaving a sheet of comparatively clear water below them which drains off rapidly and allows the solids to sink down slowly on to the bed. As the sludge dries the gases escape, leaving it more or less spongy and with an odor resembling that of garden loam.

Trickling-filter humus is brownish, flocculent and relatively inoffensive when fresh. It generally undergoes decomposition more slowly than other undigested sludges, but when it contains many worms it may become offensive quickly. It is readily digested, but when this is not done it is like activated sludge in being difficult to dry on porous beds.

430. Analysis of Sewage Sludge.—The characteristics of sewage sludge may be measured by a number of different tests, selection of which depends upon the information desired. Some of the tests which are of quite general interest in the management of sewage works have been referred to frequently in previous sections; others are important in connection with the measurement of the efficiency of sludge treatment and with particular methods of sludge disposal; still others become of value when the fertilizing constituents or grease contained in the sludge are under consideration.

Of evident significance in sewage treatment are the tests for specific gravity, moisture and volatile (organic) and fixed

(mineral) matter, together with such qualitative observations as color, consistency and odor. Specific gravity is determined by weighing a wide-mouthed flask or glass-stoppered bottle (1) empty, (2) filled with distilled water and (3) filled with sludge, the specific gravity of the sludge being given by the ratio of [(3) − (1)] to [(2) − (1)]. Moisture content and volatile and fixed matter are found in a manner analogous to the determination of total, volatile and fixed solids described in Section 253. In gaging the performance of sludge-digestion units these tests, together with certain other determinations made before, during and after treatment, may be employed. Among the additional tests there may be mentioned measurements of the H-ion concentration,[1] biochemical oxygen demand,[1] nitrogen in different forms, cellulose, fats and composition of the sludge gases. B.O.D. tests are of particular significance when sludge is to be discharged into water.

When the fertilizing value of sewage sludge is considered, the ingredients most needed in fertilizers, namely nitrogen, phosphoric acid and potash, must be taken into account. The lime content may also be significant, as well as the fat content, the latter, however, because it is apt to be an undesirable constituent which may interfere with handling the sludge and may tend to clog the soil to which it applied. To become available to plants the desirable substances, generally speaking, must be water-soluble, or if water-insoluble must be rendered available as a result of changes occurring in the soil. These changes are brought about usually as a result of bacterial activity.

The water-soluble constituents are found by extracting the dried sludge with water. The water-insoluble but active organic nitrogen, which will break down into ammonia when boiled with permanganate,[2] is considered available.[3] Nitrogen is

[1] In this connection it is of interest to note that Rudolfs has attempted to define what a well-digested sludge is in terms of its B.O.D. and pH, operators ordinarily relying on observations of the appearance, consistency and odor of the sludge. According to Rudolfs the B.O.D. should be less than 1,000 p.p.m. in 24 hour at 20°C., sludge being stable enough to be drawn when its B.O.D. is less than 1,500 p.p.m., and the pH should be higher than 7.0. These figures may have to be revised as further evidence is collected.

[2] Alkaline or neutral permanganate is used, practice varying in different sections of the country.

[3] Investigations made by Lipman and Burgess at the University of California show that different soils have markedly different capacities for utilizing the nitrogen of sludge. Their experiments were made with nine sludges obtained from Imhoff and septic tanks. The "availability" of the nitrogen, by which they consider such materials should be judged, was determined by a method originated by them. One gram of thoroughly dried, finely ground sludge was mixed with 100 grams of soil and enough water was added to produce the

reported as ammonia, 1 part of NH_3 equaling $\frac{14}{17}$ or 0.824 parts of N. Phosphoric acid occurs in three forms: (1) soluble in water and readily taken up by plants; (2) slightly soluble but nevertheless readily assimilated, in which condition it is termed "reverted"; (3) very slightly soluble and assimilated but slowly. Potash is almost wholly water-soluble.

Mineral analysis of sludge is sometimes undertaken but yields information of practical value only under special circumstances and for purposes not ordinarily encountered in sewage works practice. Grease (fats) is ordinarily determined as ether-soluble matter, the extract containing besides true grease certain gums, resins and waxes. Analytical methods are given in "Standard Methods for the Examination of Water and Sewage" and books or treatises on industrial and agricultural analysis. The student is referred to these for particulars.

431. Outline of Methods of Sludge Treatment and Disposal.—The various means of sludge treatment and disposal are presented in the following outline.[1] Treatment inherent in the process which produces the sludge, such as digestion in two-story settling tanks and concentration of sludge incident to the use of mechanical sludge-removing devices, is not considered separately.

Sludge Disposal Methods

(*A*) Disposal of untreated sludge.
- 1. Disposal in water.
  - (*a*) Dumping at sea.
  - (*b*) Dumping or discharge into great lakes, large rivers or other inland bodies of water.
- 2. Disposal on land.
  - (*a*) Flowing upon land.
  - (*b*) Covering in furrows or trenches.
  - (*c*) Lagooning.

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best moisture conditions. The tumbler containing this sample was covered with a Petri dish cover and incubated for a month at 28 to 30°C. At the end of this period nitrate determinations were made on these soil cultures. Experiments also were made to determine the average "availability" of the nitrogen in several commercial organic fertilizers.

These investigators were much encouraged by the results of their tests. In the dry climate of parts of California it may be practicable to produce air-dried sludge more easily and certainly than where there is a greater rainfall, and in this way to obtain at low cost a useful material for adding to humus-poor soils of that state. An important fact connected with the results is that the absolute quantities of nitrate produced from sludge nitrogen were often 50 to 75 per cent as high as those produced from similar weights of dried blood and high-grade tankage.

[1] Adapted from *Am. Jour. Pub. Health*, **15**, 334, and Trans., International Conference on Sanitary Engineering, 1924, p. 126.

(*B*) Treatment of sludge by one of the following methods or combination of these methods.
- 1. Digestion with or without other solids in
  - (*a*) Separate digestion tanks.
  - (*b*) Two-story tanks.
- 2. Dewatering of digested or undigested sludge by
  - (*a*) Primary removal of water with or without conditioning of the sludge by chemicals.
    - (1) Air-drying on sand beds or beds of other porous material, covered or uncovered.
    - (2) Filtering with or without additional pressure or vacuum.
    - (3) Centrifuging.
    - (4) Flotation.
  - (*b*) Secondary removal of water by heat.

(*C*) Disposal of treated sludge.
- 1. Digested but otherwise untreated sludge as under (*A*).
- 2. Sludge subjected to primary dewatering.
  - (*a*) Disposal in water.
    - (1) Dumped at sea.
    - (2) Dumped into inland waters.
  - (*b*) Disposal on land.
    - (1) Used for filling.
    - (2) Used as fertilizer.
  - (*c*) Disposal by burning.
- 3. Sludge subjected to secondary dewatering.
  - (*a*) Used as fertilizer.
  - (*b*) Used as fertilizer base.

In order to avoid repetition this outline will not be followed in detail in the following sections of this chapter. Sludge digestion, for example, already has been fully covered in connection with sewage treatment processes, and the disposal of untreated (wet) and partially dried (spadable) sludge are best covered at one time.

432. Sludge-drying Beds.—In this country the wet sludge produced by sewage treatment most commonly is run on to specially prepared sand beds on which it dries in the open air (Fig. 213). Part of the water passes into the sand and percolates through it to underdrains, while part evaporates. This method of treatment commonly has three objects in view: (1) reduction of liquid sludge to a spadable condition, suitable for disposal as fertilizer or as fill for low land, (2) reduction of the bulk of material to be disposed of, and (3) reduction of the rate of decomposition of the sludge removed from the beds, abundant moisture being a requisite to rapid bacterial activity.

Climatic conditions and the air-drying characteristics of different sludges are important considerations in providing sludge-

drying areas. The behaviors of different sludges when they are placed on sand beds have been indicated in Section 429. Estimates of the drying capacity may be made in the laboratory

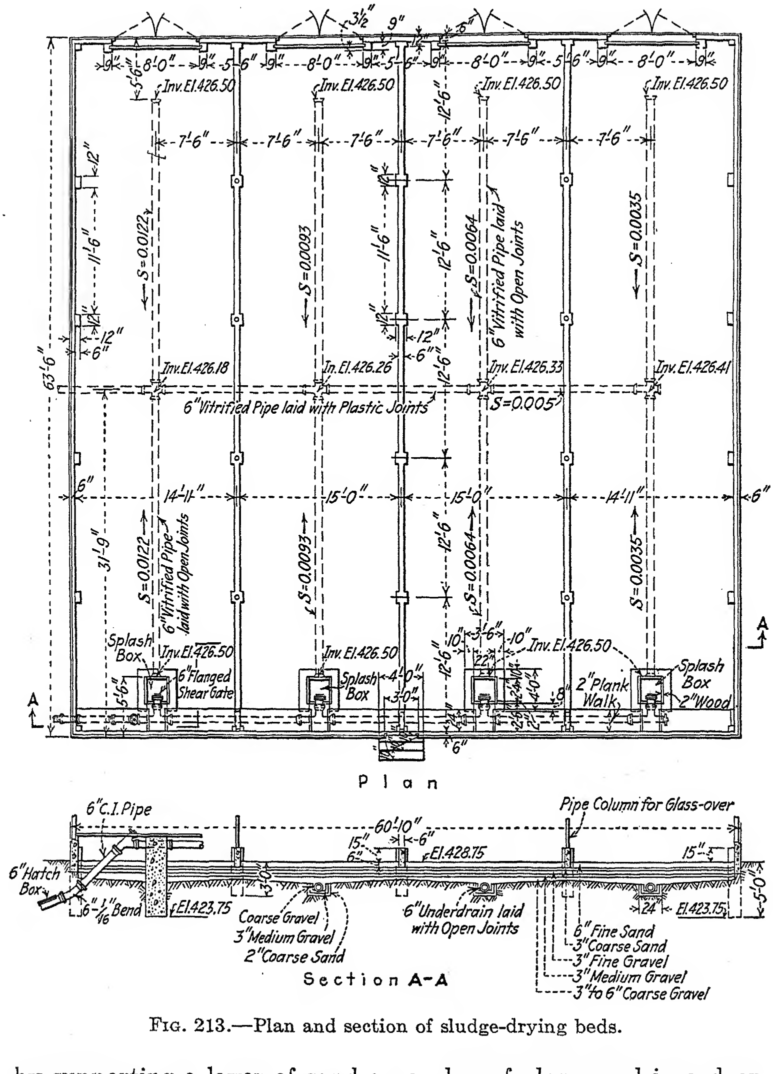

FIG. 213.—Plan and section of sludge-drying beds.

by supporting a layer of sand on a plug of glass wool in a deep glass funnel and filling the funnel with sludge.

The filtering material used in sludge-drying beds is seldom over 24 in. deep, and commonly consists of 1 to 6 in. of coarse

sand, overlying graded stone or gravel varying in size from 2 in. at the bottom to 1/16 in. at the top. In Europe cinders and other porous materials have been widely used. Open-jointed tile underdrains 4 to 6 in. in diameter laid in coarse gravel generally are provided at intervals of 5 to 20 ft. Beds are divided into compartments whose width and length are chosen so as to facilitate (1) removal of the dried sludge and (2) uniform distribution of the wet sludge on the bed. In small plants compartments are seldom over 20 ft. wide, a dimension convenient in connection with the loading of dump cars on permanent or temporary industrial tracks. The length which will generally ensure satisfactory distribution of sludge from one inlet may vary with the sludge between 50 and 100 ft.

The sludge-drying area and its subdivisions generally are surrounded by concrete curbing or earth embankments high enough to hold the maximum charge of sludge (about 12 in.), stop planks being provided at the entrance to the beds. At a number of plants the beds are glass-enclosed, the superstructure being of the common greenhouse type. The aims of this provision may be (1) to reduce the area of beds required by protecting the drying sludge against unfavorable weather conditions, (2) to permit winter drying and (3) to reduce digestion-tank requirements. The sludge liquor penetrating to the underdrains may be discharged into the plant influent or effluent, depending upon its character. The liquor is commonly clear, but may be high in bacteria. Tests at Baltimore indicate a B.O.D. almost twice that of the raw sewage.

The drying area required varies with (1) the strength of the sewage treated, (2) the character of the sludge produced and (3) local atmospheric conditions, such as rainfall, relative humidity and temperature. Allowances are generally expressed in square feet per capita, the number and depths of fillings also being given at times. It is evident from Section 429 that drying beds are particularly suitable for the dewatering of Imhoff sludge or other well-digested sludge. For use in connection with two-story tanks and separate digestion tanks treating fresh sewage solids only, open drying areas of 1/3 to 1 sq. ft. per capita are common for separate sewerage systems in the northern United States, the lower value being associated with strictly domestic sewage. Generally additional allowances of 50 and 100 per cent, respectively, are made for combined systems and excessive industrial

wastes. Usually monthly or semi-monthly fillings are possible during the summer, the sludge being run on to the beds in depths of 8 to 12 in., and becoming spadable in about 10 days during dry weather. For the southern United States and for glass-covered beds reductions of 50 per cent in the stated allowances are common. For well-digested sludges produced by combinations of two-story or separate digestion tanks with the activated-sludge process or other treatment methods, additional allowances must be made in proportion to the increase in sludge volume.

Much higher values are required for other types of sludge. It is questionable if sludge from chemical precipitation or activated-sludge tanks can be dried economically on open beds, except in small plants or in very dry climates. Both classes of sludge must be spread in thin layers in order to drain and dry, and their volume is so great that a relatively large area of drying beds must be provided and the cost of operation will be considerable.

Values varying from 3 to 7 sq. ft. per capita are given for activated sludge and sludge from chemical precipitation tanks. At Brockton, sludge from plain sedimentation tanks is dried on beds formerly used as intermittent sand filters, 2 ft. of fine sand having been replaced by coarse sand. The area provided is about 2 sq. ft. per capita. At Tenafly, N. J., glass-covered sludge-drying beds, designed to serve 7,500 persons with an area of 1 sq. ft. per capita, are reported to dry activated sludge satisfactorily in 4 or 5 days. In 1929 these beds were handling the sludge from a population of 4,000.

After sludge has dried sufficiently to become spadable it must be removed from the beds and disposed of. Generally in small plants the sludge is spaded or forked into wheelbarrows or dump-cars for removal to the disposal site. In large plants power-driven equipment may be used.

433. Sludge Conditioning.—Before taking up the dewatering of sludge by filtering it through cloth, brief consideration may be given to some of the factors entering into the problem. These are well illustrated by investigations conducted by Copeland, Heisig and Wilson at Milwaukee and by Pearse and Mohlman at Chicago on the dewatering of activated sludge. At Milwaukee it was found that with sewage temperatures of about 45°F. the time required to dewater a given amount of excess sludge produced at this low temperature was about 20 times as long as when the temperature of the sewage was about 70°F. The reason for this

difference appeared to lie chiefly in the greater dispersion of particles in winter sludge. The finely divided matters tended to pass into the interstices of the cloth and "blind" it, preventing the passage of water. From a consideration of the principles of colloid chemistry three methods of sludge conditioning which ordinarily result in increasing the size of sludge particles suggested themselves, (1) changing the H-ion concentration of the sludge, (2) adding coagulants to the sludge, and (3) heating the conditioned sludge.

As produced, the sludge particles were found to be negatively charged, the pH value being about 7.4. It was next realized that much of the sludge consisted of protein matters which assume a positive electrical charge in strongly acid solutions and a negative one in strongly alkaline solutions. Since with increasing electrical charge particles tend to break up into smaller parts, while they tend to gather together into larger units when the electrical charge is reduced, the best condition would be reached when the electrical charge equals zero (at the so-called "isoelectric point"). When sulphuric acid was added to reduce the negative electrical charge it was found that the isoelectric point of Milwaukee sludge occurred at a pH value of 3.4. Sludge so treated filtered at five times its normal winter rate. This being insufficient to make up for the difference in rate of filtration of summer and winter sludge, aluminum sulphate, a substance or electrolyte furnishing colloidal alumina carrying a high positive charge (opposite to that of the sludge) was next added to the sludge and the sludge brought to the isoelectric point with sulphuric acid, combined use of alum and H_2SO_4 being cheaper than using alum alone. With the quantities of alum employed per 100 gal. of 98-per cent sludge (1.6 lb.) the isoelectric point was reached at pH 4.4, and the sludge was found to filter at eight times its normal rate. This rate was still too low, and since coagulation at the isoelectric point is accelerated by the application of heat, the conditioned sludge was heated to 180°F. It was then learned that at this temperature acidified sludge filtered at 30 times the normal rate and acid- and alum-treated sludge at 40 times the normal rate.

Untreated sludge filtered poorly when heated, as did septic sludge. The percentage increase in filtration due to conditioning was furthermore shown to be independent of the initial rate. The effects of the different methods of sludge conditioning upon

sludge filtration at Milwaukee are clearly illustrated in Fig. 214 for different seasons of the year.

Investigations, originally at Chicago and later at Milwaukee, showed that chlorinated copperas and ferric chloride were particularly good electrolytes. Such electrolytes have been employed successfully without acid. Lime was employed many years ago in conditioning sludge from chemical precipitation tanks and septic tanks. Acidifying with phosphoric acid, which is of fertilizing value, has proved to be uneconomical at Milwaukee, because

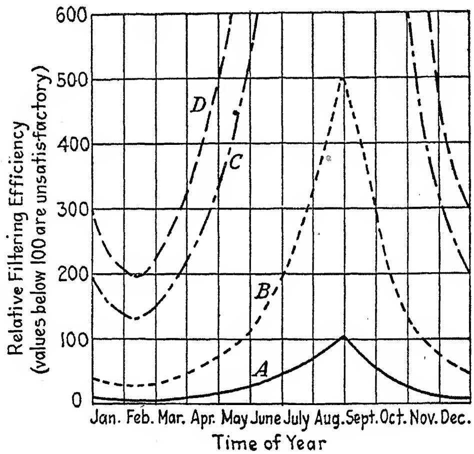

FIG. 214.—Seasonal changes in the relative filtering efficiency of activated sludge at Milwaukee when the sludge is conditioned in various ways.

A. Cold sludge—untreated.
B. Cold sludge treated with H_2SO_4 to make pH = 3.4.
C. Acidified sludge heated to 190°F.
D. Sludge treated with alum and acid to make pH = 4.4 and heated to 190°F.

much of the acid passed off into the filtrate. Use of sulphurous acid has produced objectionable operating difficulties. In practice, furthermore, sludge will seldom be heated to as high temperatures as employed in the Milwaukee tests.

At Plainfield alum has been successfully added to incompletely digested sludge in order to improve its drying on open sand beds. The alum is added to the sludge just prior to running it onto the drying beds. Well-digested sludge is said not to need this treatment.

Activated sludge obtained from plants producing a well-nitrified effluent apparently is dewatered more readily than

excess sludge from works with a low-grade effluent, the dividing line being in the vicinity of 5 p.p.m. of nitrate nitrogen.

The most successful method of sludge conditioning can be determined only by extensive experimentation. Laboratory tests are useful in making preliminary studies, a Buechner funnel[1] being employed to support the filtering medium. When the fertilizing value of the sludge is of importance the loss of nitrogen, phosphoric acid and potash in the filtrate must be taken into account.

Separation of solids and liquids by flotation, which may be considered both a conditioning and treatment method, has not been very successful. Acid is added to the sludge in small quantities, in an attempt to coagulate the solids and float them to the surface where they are removed. The underlying liquid, however, generally retains large quantities of solids.

434. Sludge Filters.—Aside from largely unsuccessful attempts at the filtration of sludge by gravity, through fine wire mesh, filtros plates and absorbent material, such as garbage tankage, three types of filtering devices may be mentioned, in all of which changes in pressure (above or below atmospheric) are employed to accelerate filtration. These devices may be classified as follows: (1) filters in which the sludge is put under pressure, (chamber or leaf filter press), (2) filters in which bags containing the sludge are squeezed (bag press), and (3) filters in which the sludge is dewatered by suction (vacuum filters). The construction of typical sludge filters is shown in Fig. 215.

The chamber or leaf press, employed widely in the chemical industries, usually consists of cast iron plates covered with cloth, such as 11-oz. army duck. The covered plates are hung in a frame commonly equipped with a fixed and a movable head. The plates are forced together, a space being left between the cloth surfaces on the plate faces and sludge being pumped through a central opening in the plates. The sludge covers the cloth and the liquid is forced through the cloth to the plate surface whence it enters drainage holes which carry it away through openings in the bottom of the plates. After the presses are filled and drainage is completed, the sludge valve is closed; the movable head is then pulled back and the cake drops out. There are a number of different makes of presses. Pressures of 80 to 120 lb.

[1] This is a porcelain funnel with a flat perforated plate laid in the bottom of a cylinder above the sloping sides.

per sq. in. are employed. Breakage of cloth was observed at Chicago when the higher pressure was applied.

Burlap is employed generally as the filter cloth in bag filters, an 8- to 12-oz. grade being used. The bags are filled from the top or bottom with sludge. They are suspended between drainage sheets and squeezed by large platens worked by a toggle joint or direct pressure. The use of Berrigan and Worthington

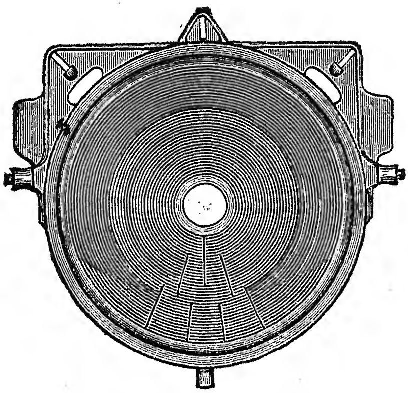

Plate used in filter press

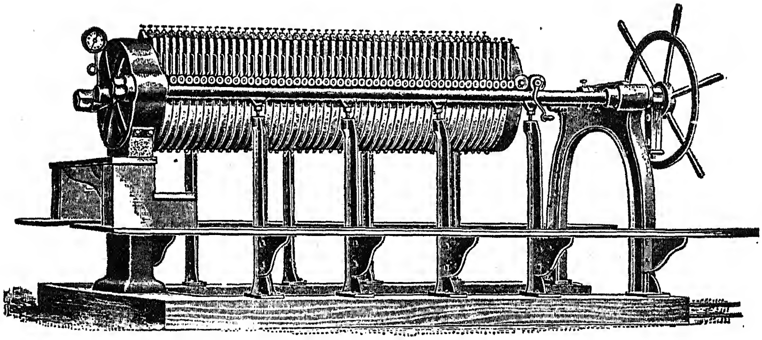

FIG. 215*a*.—Chamber or leaf filter press.

presses has been studied at Chicago for dewatering activated sludge. Hydraulic pressure is employed.

In the Oliver type of vacuum filter, employed at Milwaukee, a rotating drum of copper wire mesh is covered with cloth wired on to the drum with No. 14 wires spaced 1 in. apart. The drum is divided into sections to which a vacuum or pressure may be applied. Operation is continuous. The drum passes through a basin filled with sludge, the area submerged being adjustable

between 15 and 40 per cent. Agitators, working in the basin, keep the sludge solids from settling. As each section passes

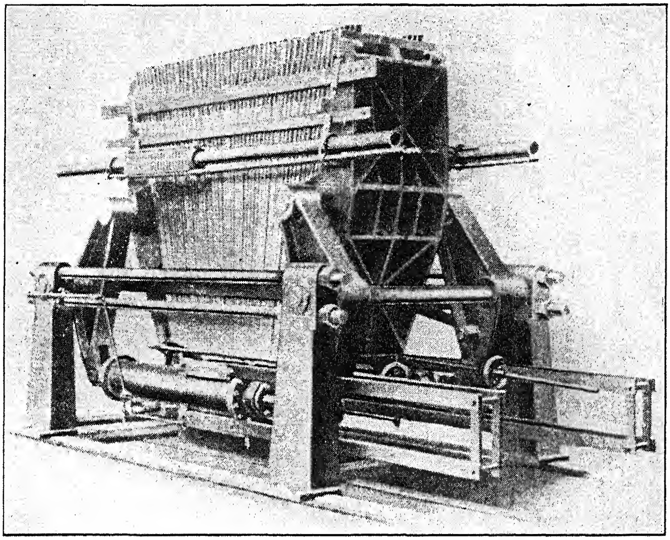

FIG. 215b.—Bag press (Worthington).

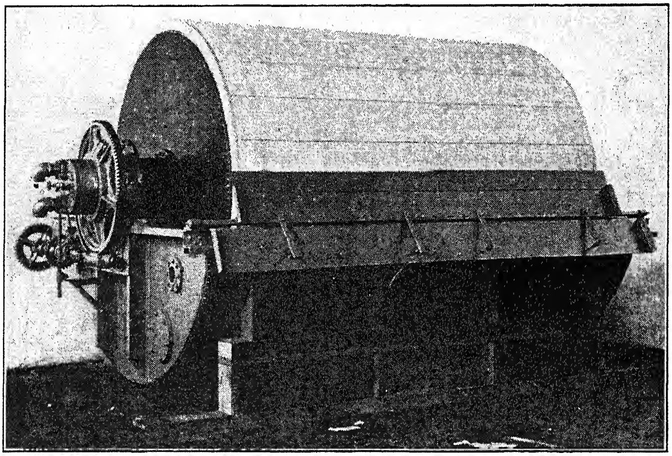

FIG. 215c.—Rotary drum vacuum filter (Oliver).

through the sludge a vacuum is applied which will pick up the desired thickness of sludge, a coating of 1/8 to 3/16 in. being usual. Suction is then continued, the vacuum commonly being increased

until the section is about to re-enter the sludge. At this point in the travel of the drum a slight pressure is applied to force the sludge from the cloth and a scraper removes the dewatered coating. A vacuum of 11 in. is employed at Milwaukee while the cloth passes through the sludge, and is increased to 22 in. during the remainder of the filtering process. The cloth may be cleaned from time to time by brushing, by steam or by a caustic solution. In the American type of filter rotating leaves, supported on a hollow shaft, are employed.

There has been much experimentation with different cloths for vacuum filters, cotton duck, Canton flannel and woolen cloth having been used on a large scale. At the present time (1929), Canton flannel is deemed most satisfactory.

All three types of filters have been tested at Chicago and Pearse reports the results shown in Table 105. The character of the sludge should be taken into account when comparisons are to be made.

TABLE 105.—OPERATING RESULTS OF FILTER PRESSES DEWATERING ACTIVATED SLUDGE

Press or filter	Filter area, sq. ft.	Cake produced		Cycle, hours
		Lb.	Moisture, per cent	
Simplex leaf filter	1,815	4,400	78	7
Worthington bag filter	1,440	3,500	76	5
Berrigan bag filter	3,520	10,000	78	8
Oliver vacuum filter	495	1,400[1]	80	Continuous

[1] Per hour.

The life of filter cloth seems to vary between 1 and 2 months. The sludge liquor is generally offensive and usually is returned to the treatment plant.

435. Sludge Centrifuges.—In this country centrifugal separation of sludge liquor and solids has been attempted only on an experimental basis. A number of German plants have had centrifuges in operation for several years. Encouraging results were obtained in tests at Baltimore during 1924-25.[1] Working with semi-digested sludge and a solid-drum type of automatic centrifuge, an average removal of 65.1 per cent of the sludge

[1] *Sewage Works Journal*, 1929; **1**, 120.

solids was accomplished, the moisture of the cake being 71.1 per cent. The machine revolved continuously at 1,200 r.p.m., sludge being admitted (inlet period), cut off (post-centrifuge period), and the cake discharged. The inlet period lasted 7.6 min. and the post-centrifuge period 3.7 min. The wet sludge averaged 93.9 per cent moisture and 68.9 per cent volatile matter. An average inlet rate of 40.1 gal. per minute was maintained, 11,800 gal. being handled every 8 hours with a total current consumption of 295 k.w.h. The average solids content of the sludge liquor was 2.7 per cent, and its B.O.D. 9,100 p.p.m. The latter could be reduced to 2,815 p.p.m. by adding 1.75 lb. of alum per 100 gal. of effluent. The offensive character of the sludge liquor is considered one of the disadvantages of the process. At Baltimore it was calculated that addition of the sludge liquor to the untreated sewage would increase the B.O.D. of the latter by 4.3 per cent. Adding sludge liquor to trickling-filter effluent in the ratio of 1 to 1,250 reduced the relative stability of the effluent from 96 to 84 per cent.

436. Heat Dryers.—If sludge is to be dried to serve as a fertilizer or fertilizer base, for which purpose a practically dry material is required, rotary dryers such as are used in many industrial processes may be employed. Dryers of the direct or indirect type may be used and will reduce the moisture to 10 per cent or less, the maximum value commonly specified for commercial fertilizers. Before being introduced into the rotary dryer the sludge generally must be dewatered by primary means to 80 or 85 per cent moisture.

At Milwaukee six Atlas dryers of the direct-indirect type, 60 ft. long and 7 ft. in diameter, are used, each producing daily 10 to 15 tons of fertilizer with 4 per cent moisture from activated sludge dewatered to 82 per cent moisture content. An equal quantity of dry material is mixed with the dewatered sludge before it enters the dryers in order to reduce the initial moisture content and prevent caking. The temperature of drying ranges from about 1,000°C. at the inlet end to 500°C. at the outlet. The gases from the dryers must be passed through dust chambers or dust arresters (cyclones) and sometimes through washers, in order to remove the fine dust. To avoid causing offense the gases may also have to be deodorized by additional treatment such as washing. Chlorine treatment is employed at Milwaukee at times.

Complete dewatering of sludge necessitates the use of much conveying machinery. The wet sludge is readily pumped, but sludge paste from vacuum filters balls together unless suitably handled. At Milwaukee the use of metal band conveyors has been most successful. The dried sludge to be sold for fertilizing purposes generally is ground and bagged and suitable machinery must be provided to handle, grind and bag the material. Storage space also must be available and the sludge may have to be classified according to its nitrogen content.

437. Final Disposal of Sludge.—Generally speaking there are two broad methods of final sludge disposal which correspond to the disposal of the liquid sewage by dilution and irrigation. They are (1) disposal in water and (2) disposal on land, treatment sometimes preceding disposal, sometimes not.

Disposal in Water.—A number of the seacoast towns of Great Britain carry the sludge to sea and dump it in deep water. In this country, the city of Providence, R. I., and the sewerage district of the Passaic Valley, N. J., conform to this practice, and plans for treatment works for a part of New York City call for its adoption. Treatment of the sludge prior to disposal is not essential, except for the purpose of reducing the volume to be handled. At Providence, the sludge from chemical precipitation tanks is treated with lime and dewatered to a moisture content of about 75 per cent. The sludge cake is placed in a double-bottomed scow which is towed to the disposal area about 14 miles distant from the plant. Sludge from the plain sedimentation tanks of the Passaic Valley sewerage system is pumped into scows which dump it at a distance of 12 miles from the shore.

At Cleveland, Imhoff sludge is pumped into Lake Erie; and several American communities discharge sludge into inland streams when they are in flood.

Disposal on Land.—Wet sludge may be disposed of on land by (1) flowing it on to the land, (2) running it into trenches which are covered after being filled, and (3) discharging it into lagoons. Dried sludge may be used for filling or for fertilizer purposes.

Disposal of sludge by flowing it on to land or running it into trenches and covering it requires a large disposal area. Disposal of undigested sludge in this manner is not regarded with favor in this country, on account of the odors produced and the sanitary hazards involved. Trenches about 3 ft. wide and 18 in. deep have been employed at certain English disposal works where the

area required has been found to be from ¼ to 1 acre per 1,000 long tons of sludge. As the land once used in this way cannot be employed again for the purpose for a long period, a very large tract must be available for trenching.

Lagooning of wet sludge has probably been employed more widely than any other disposal method. A lagoon is a basin formed by surrounding a tract of land by a dike of earth. Lagoons vary greatly in size, some being a fraction of an acre and others several acres in extent. They are filled with sludge to a depth of 1 to 5 ft. The sludge is left to dry by such losses of water through the soil as may take place naturally, and by evaporation. Drying may take from 2 to 6 months and usually is accompanied by offensive odors when undigested sludge is disposed of in this way. Digestion naturally takes place in the lagoons. Lagoons may be worked until they are completely sludged up, or the drained and digested material may be removed from time to time and used to fill low-lying areas in the neighborhood. At Toronto little trouble is reported from the operation of sludge lagoons receiving undigested sludge, as long as the scum forming on the lagoons remains undisturbed. The Toronto lagoons are worked in series, one sludging up while the overflow passes through two other lagoons before discharging into a bay of Lake Ontario. At Syracuse, N. Y., sludge from plain sedimentation basins is removed continuously by Dorr mechanisms and pumped to a disposal area about two miles distant where it is lagooned together with many times its volume of industrial wastes from the Solvay Chemical Works. The resulting mixture is completely sterilized by the chemicals in the Solvay wastes and the overflow from the lagoons flows into Onondaga Lake.

Sludge from drying beds can be burned if mixed with coal or other fuel, but generally an attempt is made to get rid of it by dumping it on low land as a waste material without value. In a few places it is hauled away by farmers as a low-grade fertilizer, and probably a greater use of the material in this way could be accomplished if persistent endeavor were made to interest farmers in its value. Marked success has been reported from Schenectady where an educational program has been carried on for a number of years by Cohn.

438. Hygienic Considerations in Sludge Disposal.—The disposal of sewage sludge, and more particularly its use for fertilizing

purposes, raises the question of the danger of spreading typhoid and other diseases by this means. Tests by Kligler[1] showed that the typhoid bacillus may survive in solid feces for about 10 days and in liquid feces for about 6 days. In moist soil the time varied from 30 to 70 days, depending on the pH value of the soil, being shorter for acid soils than for alkaline ones. In dry soil the viability was reduced to 20 days and the time of survival in septic tanks varied from 5 to 14 days. Other intestinal bacteria were shown to be less resistant.

A study of available data indicates, as would be expected, that generally speaking, fresh solids are more dangerous than well-digested sludge and wet solids more so than partially dried solids. Dry solids are relatively safe, and heat-dried solids more so. Hygienic considerations require, therefore, as in the case of sewage farming, that sewage sludge, except heat-dried sludge, should not be brought into contact with produce which is consumed without being cooked.

439. Fertilizing Value of Sludge.—The utilization of sewage sludge as a fertilizer has received a marked impetus since the introduction of the activated-sludge process, which produces a sludge containing from two to four times as much nitrogen as other sewage sludges with the exception of the humus from trickling filters. The fertilizing constituents contained in sewage sludges obtained by different treatment methods are shown in Table 106. Since the figures given in this table do not yield information in a form in which they are sufficiently significant from the standpoint of sludge use for fertilizing purposes, a typical fertilizer analysis of activated sludge from Milwaukee, reported by Kadish,[2] is shown in Table 107.

The three ingredients most needed in fertilizers, nitrogen, phosphoric acid and potash, are dissolved so easily that in the case of commercial fertilizers they are not applied until just before they are needed. On the other hand, in the case of barnyard manure a portion of those elements useful to stimulate plant growth is so combined with other substances as to require breaking down into simpler and more easily soluble matter through bacterial action, thus rendering it available for plant food. Hence farmers sometimes apply manure to their fields in the autumn in order that its valuable ingredients may be made

[1] Rockefeller Monograph No. 15, 1921.

[2] Trans., International Conference on Sanitary Engineering, 1924; p. 134.

TABLE 106.—RESULTS OF SLUDGE ANALYSES

Type of sludge	City	Per cent volatile matter	Per cent, dry basis				Reference
			Total nitrogen as N	Phosphates as P_2O_5	Potassium as K_2O	Ether-soluble matter	
Septic tanks	Worcester, Mass.	43.9	3.01	1.8	...	....	"American Sewerage Practice," Vol. III, pp. 470–471
Septic tanks	Cleveland, Ohio	40.8	1.50	1.2	...	9.0	HOFFMAN, PRATT and HOMMON, Report on Tests at Sewage Tr. Sta., Cleveland, Ohio, 1914
Septic tanks	Waterbury, Conn.[1]	45.5	1.35	...	...	10.8	"American Sewerage Practice," Vol. III, pp. 470–471
Imhoff tanks	Columbus, Ohio	....	2.8	1.6	0.6	24.4	*Public Health Bull.* **132**
Imhoff tanks	Fitchburg, Mass.	70.4	2.0	1.6	0.6	7.7	*Public Health Bull.* **132**
Imhoff tanks	Rochester, N. Y. (Irondequoit)	47.6	2.2	0.8	0.6	8.0	*Public Health Bull.* **132**
Activated-sludge process	Houston, Tex.[2]	44.7	3.0	2.1	0.4	5.0	*Public Health Bull.* **132**
Activated-sludge process	Milwaukee, Wis.[3]	57.5	4.5	2.0	...	5.4	Third Annual Rept., Milwaukee Sewerage Commission
Activated-sludge process	Sherman, Tex.	76.6	5.8	2.8	0.6	10.4	"American Sewerage Practice," Vol. III, pp. 470–471

[1] Average of two analyses.
[2] From dryer.
[3] From sedimentation tank.

available and may leach into the soil by the time they are needed in the spring. Sewage sludge is similar to barnyard manure in these respects.

Table 107.—Typical Analysis of Milwaukee Activated Sludge

	Per cent
Moisture	6.20
Total nitrogen as ammonia	7.42
Water-soluble nitrogen as ammonia	1.13
Active water-insoluble organic nitrogen as ammonia (alkaline permanganate method)	4.17
Availability of water-insoluble organic nitrogen	66.35
Total availability of nitrogen	71.48
Total phosphoric acid	2.36
Potash (water-soluble)	0.13
Fat	4.87

Many practical tests of the fertilizing value of the sludge from plain sedimentation and chemical precipitation tanks have been made in Great Britain. In a general way they indicate that such sludge is of little value on grass land or as a fertilizer for root crops of rapid growth, but it has increased the yield of wheat 10 to 12 per cent in some cases. Imhoff sludge and other well-digested sludges have produced excellent results in California in comparison with certain organic fertilizers. Humus from trickling filters appears to be a good fertilizer. The best results and most complete information, however, have been obtained for dried activated sludge. The high nitrogen content of activated sludge as well as trickling-filter humus is commonly ascribed to the inclusion in the sludge of much of the nitrogen lost in the effluent of other treatment processes.

Utilization of activated sludge has been investigated particularly well at Milwaukee where comparative tests with commercial fertilizers have shown that activated sludge yields good results as a fertilizer for lawns, farm products and greenhouse products. For use on lawns and golf courses it is considered superior to sheep manure which is widely applied to grass areas, and has the advantage of not burning the grass or developing objectionable odors. The dried sludge is not a complete commercial fertilizer since it contains insufficient proportions of phosphoric acid and potash. It may be used, however, in combination with these substances as well as with ammonium

sulphate. The latter, being immediately available, is often employed to furnish the initial stimulus for plant growth, the sludge serving as humus and as a continuing source of plant food after the ammonium sulphate has been utilized.

If grease is present in sludge in such proportion as to be seriously detrimental to the fertilizer, it may be necessary to degrease the dried sludge. The best way to do this appears to be by one of the solvent extraction processes used in garbage reduction, a process somewhat dangerous and not likely to prove advisable unless the sludge contains an abnormally large proportion of grease, likely to be the case only where certain industrial wastes high in fats reach the sewers.

440. Commercial Utilization of Sludge as Fertilizer.—Many attempts have been made to sell dried sludge as a fertilizer in Europe, probably the most successful being that at Glasgow, Scotland, where all the sludge produced at the Dalmarnock works is sold in the form of pressed cake or dried fertilizer. This commercial success was ascribed by the Royal Commission on Sewage Disposal to careful business organization and judicious advertising; the commission estimated that the sales of sludge decreased the net cost of sewage treatment about 80 cents per 1,000,000 gal. Investigations of the practicability of drying sludge to form a fertilizer base have generally been discouraging in the United States. Since the advent of the activated-sludge process there has been a somewhat brighter prospect for sludge utilization at large treatment plants.

From what has been said it will be seen that the preparation of fertilizer from activated sludge is a manufacturing process having a number of stages which probably can be conducted profitably only on a large scale. In addition to this production aspect of the work, there is a business side involving the sale of the product at the highest possible price and the management of the entire enterprise in an efficient and economical manner. Whether it is desirable for a municipality to embark upon such a business venture will depend upon the local conditions. Unless the enterprise is handled efficiently there are possibilities of grave financial troubles. The overhead expense in any case is likely to be large, and the proportionate cost of operating a small sludge fertilizer plant will probably be much greater than for large plants.

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Problems

1. Estimate the drying bed area required in connection with a battery of Imhoff tanks treating on an average 5 m.g.d. of average sewage in the northern United States, (*a*) if the sewerage system is on the separate plan and (*b*) on the combined plan.

2. Imhoff sludge containing 84 per cent moisture is run onto sludge-drying beds. Estimate the cu. ft. of spadable sludge obtained from 1,000 gal. of wet sludge.

3. How many gallons of 98-per cent activated sludge can be handled per hour, on the basis of the results reported by Langdon Pearse, by means of an Oliver filter 8 ft. in diameter and 10 ft. long? *Ans.* 850 gal.

4. Estimate the filter area required to dewater the excess sludge produced by an activated-sludge plant treating 10 m.g.d. of average sewage. Consider the use of (*a*) leaf filters, (*b*) bag filters and (*c*) vacuum filters.

5. Estimate the tons of heat-dried activated sludge produced by a plant treating 20 m.g.d. of average sewage.

CHAPTER XX

CHLORINATION OF SEWAGE, DISPOSAL OF INDUSTRIAL WASTES, AND CHOICE OF SEWAGE TREATMENT AND DISPOSAL METHODS

CHLORINATION OF SEWAGE

441. Development of Sewage Chlorination.—The outstanding work on the disinfection of sewage by chlorination was done by Phelps[1] at Boston, Red Bank, N. J., and Baltimore in 1906–1907. Prior to this time chlorine, chiefly in the form of hypochlorites, had been studied both as a general disinfectant and as a deodorant. Its economical and practical utilization, however, for the destruction of pathogenic organisms as well as other living organisms in sewage, had not been demonstrated.

In this country the chlorination of sewage is finding increased use for the purpose of protecting water supplies, bathing beaches and shellfish layings situated at relatively short distances from sewer outfalls. Recent investigations indicate, furthermore, that chlorination will reduce to some extent the biochemical oxygen demand of sewage. Chlorination has also been employed to advantage in controlling odors at treatment works, and reference has been made in previous chapters to its use for this purpose in connection with trickling filters and the heat-drying of activated sludge. The combatting of filter flies by chlorination has also been touched upon.

442. Chlorine and Chlorine Compounds.—The disinfection or deodorizing of sewage is generally accomplished by chlorine in one of the following forms: (1) liquid chlorine, (2) bleach, and (3) electrolytic chlorine. All of these have their origin in the electrolysis of salt brine. The reactions may be indicated as follows:

2×58		2×18		71		2×40		2
$2NaCl$	$+$	$2H_2O$	$\rightleftarrows$	Cl_2	$+$	$2NaOH$	$+$	H_2
Sodium chloride (salt)		Water		Chlorine (gas)		Sodium hydroxide (caustic soda)		Hydrogen (gas)

[1] *Water Supply Paper* **229**, U. S. Geological Survey, p. 60.

The chlorine gas may be (1) removed as produced, dried and liquefied by pressure to form "liquid chlorine"; (2) passed over slaked lime to form "chloride of lime" or "bleaching powder"; or (3) utilized as "electrolytic chlorine" at the point of generation, in the form of chlorine gas when the products of electrolysis are separated, or as sodium hypochlorite when they are permitted to react upon one another.

In this country liquid chlorine and bleaching powder are by-products of the manufacture of caustic soda, while sodium hypochlorite is usually generated in chlorine cells at the point of application.

Liquid chlorine is commercially available in steel cylinders holding 100 or 150 lb. of chlorine, in drums of 1 ton net, and in single- or multi-unit tank cars holding 15 tons of gas. The chlorine is under a pressure varying from about 40 lb. per square inch at the freezing point of water to 60, 100 and 140 lb. per square inch at 50, 75 and 100°F., respectively. The solubility of the gas in water is high, decreasing from 14,600 p.p.m. at 32°F. to 10,000, 6,600 and 4,200 p.p.m. at 50, 75 and 100°F., respectively, when pure water is exposed to an atmosphere of pure chlorine and the combined pressure of the gas and water vapor is one atmosphere. Bleaching powder comes on the market in iron drums holding 10, 50, 100, 300 or 700 lb. The chief constituent of bleaching powder is calcium chloro-hypochlorite ($CaClOCl$), the active substance in chlorination by bleach. Good bleach contains about 65 per cent of $CaClOCl$, which forms in water calcium hypochlorite [$Ca(OCl)_2$] and calcium chloride ($CaCl_2$). Through established commercial usage, the strength of bleaching powder is reported in terms of "available chlorine," which is the quantity of chlorine liberated on decomposing the hypochlorite with acid. It is commonly determined by finding the quantity of iodine liberated from potassium iodide (KI).[1] A study of the reactions of chlorine and hypochlorites in water, given in Section 327, will demonstrate that in what ever form chlorine is applied, two atoms of chlorine are required to liberate one atom of oxygen, and that the "available chlorine" is therefore a measure of the oxidizing power of chlorine or hypochlorite. Since the ratio of the chlorine content to the total weight of $CaClOCl$ is $^{71}/_{127} = 0.56$, the available chlorine is given by

[1] $2CaClOCl$ in water $\longrightarrow Ca(OCl)_2 + CaCl_2$

$Ca(OCl)_2 + 4KI + 4HCl \longrightarrow CaCl_2 + 4KCl + 2I_2 + 2H_2O$, or 4 atoms of iodine for 4 atoms of chlorine in the bleach.

multiplying the per cent of CaClOCl in the bleaching powder by 0.56. With 65 per cent pure bleach, for example, the available chlorine is $0.56 \times 65 = 36.5$ per cent. A value of $33\frac{1}{3}$ per cent is common. Since chlorine gas is practically pure, its available chlorine is nearly 100 per cent. Bleaching powder absorbs moisture and loses chlorine on exposure to the air, with a concurrent loss of its disinfecting power. The use of liquid chlorine has superseded in large measure the formerly wide use of chloride of lime.

A number of different cells have been constructed to produce chlorine or sodium hypochlorite (commonly the latter) at or near the point of chlorination. In the electrolysis of salt water, sodium is liberated at one pole and chlorine at the other. The sodium combines with the water to form sodium hydroxide and hydrogen. The chlorine, unless removed as gas, combines with the former and produces sodium hypochlorite while the hydrogen escapes. Side reactions reduce the quantity of chlorine or hypochlorite produced below the theoretical value and increase the power consumption. Theoretically the power needed to produce 1 lb. of available chlorine is 1.23 kw.-hours and the quantity of salt is $^{58}/_{35} = 1.65$ lb. Actual power and salt requirements, however, are respectively about 2 and 3 times the theoretical quantities. Electrolytic chlorine as yet has not found favor in sewage treatment works, and there are no important installations in which chlorine cells are used.

443. Methods of Chlorination.—Liquid chlorine apparatus now is employed generally in large or permanent installations, while bleaching powder may be employed where sewage is to be chlorinated on a small scale or as a temporary expedient.

Liquid Chlorine.—Liquid chlorine may be applied to sewage in one of two ways, designated as (1) direct feed and (2) solution feed. In the direct-feed method, measured quantities of chlorine gas are diffused through the sewage and are quickly taken into solution. In the solution-feed method the gas is first dissolved in water and the chlorine solution then is added to the sewage. The quantity of chlorine commonly is controlled by (1) regulating the pressure drop of the gas across a fixed or variable orifice or (2) adjusting the intermittent pulsating displacement of a definite volume of water by chlorine gas or vice versa, or by regulating the rate of bubbling the chlorine through water. The first method generally is employed for measuring large

quantities of chlorine, the second for small ones. The rate of dosage generally is checked by recording the loss in weight of the chlorine container in a definite period of time. Recording scales have been developed, as has apparatus which will proportion the rate of feed to the sewage flow. Since the quantity of chlorine

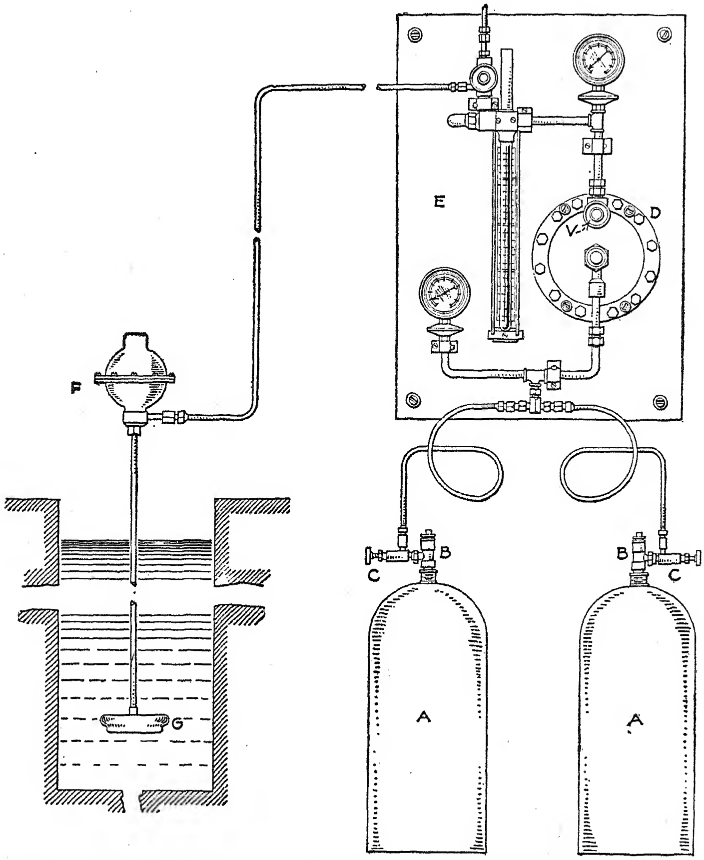

FIG. 216.—Manually controlled direct-feed chlorinator (Wallace & Tiernan Co.).

required depends upon the character as well as the volume of sewage, as will appear in Section 444, manual operation is generally more satisfactory. Where sewage is pumped arrangements can be made to start and stop the chlorinator and the pumps at the same time.

A manually controlled apparatus made by the Wallace & Tiernan Co. for treating sewage with chlorine gas or a so-called solution of chlorine

made from liquid chlorine is shown in Fig. 216. The tanks of liquid chlorine, *A*, *A*, generally are placed on scales so that the variation in their weight as their contents are removed can be measured and in this way a check obtained on the rate of flow indicated by the chlorinating apparatus. Each tank has a tank valve, *B*, *B*, to which the operator attaches an auxiliary tank valve, *C*, *C*, which is the main shut-off valve during operation. When the valves are opened the liquid chlorine in the tank gradually becomes a gas and passes from the tank to a compensator, *D*, which maintains a constant flow of the gas irrespective of the pressure in the tank, so long as the setting of the control valve, *V*, forming part of the compensator, is not changed. The rate of flow of the chlorine is changed by altering the setting of this control valve.

From the compensator, the gas passes to a meter, *E*, which is a manometer having a scale calibrated to show the flow of chlorine gas in pounds per 24 hours or other unit desired by the purchaser. From the meter the gas flows to the place of application through a copper or galvanized-iron pipe which should not be over 500 ft. long. The gas is delivered through a chlorine check valve, *F*, which is necessary to prevent moisture from the sewage passing back to the chlorinator and interfering with its operation. From the check valve, the gas passes through a silver tube to the diffuser, *G*, submerged at least 4 ft. in the sewage. This diffuser is composed of a composition sponge of fine porosity held in a non-corrodible casing. The sponge is kept saturated with water by capillary action of the material of which it is composed, and the chlorine in passing through the sponge becomes thoroughly mingled with the water and is discharged in this condition into the sewage.

Use of solution-feed chlorination, while generally more satisfactory in operation, is dependent upon a supply of water under adequate pressure to operate the auxiliary apparatus. Diffusers used on direct-feed installations must be submerged to a sufficient depth (commonly 4 ft. or more) to prevent escape of chlorine.

Chlorine gas is extremely corrosive in the presence of moisture, and gas lines, valves and pressure apparatus must, therefore, be kept free from moisture or be constructed of resistant materials, such as silver, hard rubber or glass. At low temperatures chlorine hydrate is sometimes formed in the measuring apparatus. Under these conditions heat must be supplied during cold weather to the apparatus or to the chlorinating room or house.

The liquid chlorinating apparatus should be placed in a well-ventilated room, so that in case there is any escape of gas it can be removed immediately. This room should open out of doors. The treatment of a person affected by the gas is to give fresh air and ether; the latter must be administered carefully, and only in sufficient amount to relieve pain.

Bleaching Powder.—Where bleaching powder is to be employed instead of liquid chlorine, provisions must be made to (1) dissolve

the powder and (2) measure the volume of solution added to the sewage. Naturally the strength of the solution must be known. One or more mixing tanks, solution tanks and feed tanks must be provided. The materials employed for the construction of these tanks may be wood, tile, concrete or cast iron. The bleach is first dissolved in the mixing tanks, which commonly should hold 4 or more gallons of water for each pound of bleaching powder. Mixing may be done by hand, or by a paddle stirring-

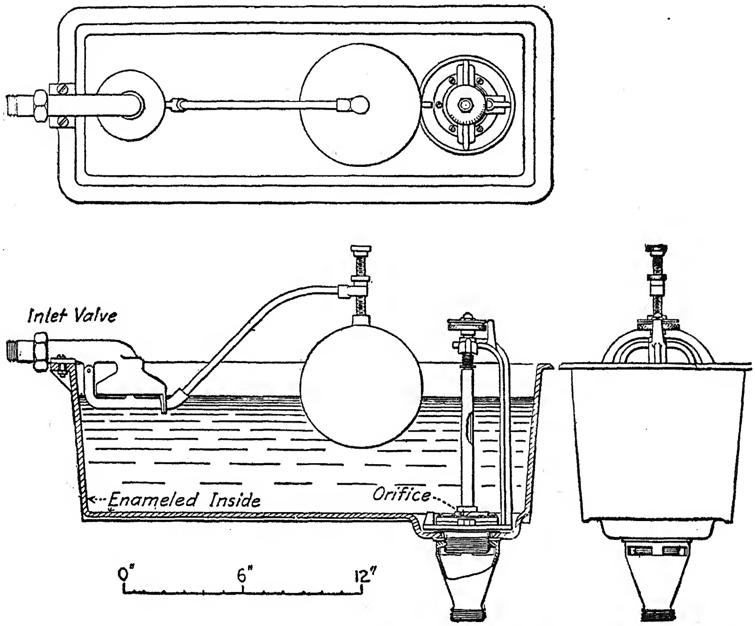

FIG. 217.—Orifice feed tank (Pittsburgh).

mechanism. From 12 to 24 hours are required to ensure complete dissolving and clarification of the solution. The inert constituents of the bleach are drawn off through a drain pipe, the clear solution being led to the solution tank, where it generally is diluted to a strength of 1 or 2 per cent and stirred to maintain uniform strength. The hypochlorite solution may be fed to the sewage from an orifice feed tank in which a constant head is maintained on an adjustable orifice (Fig. 217). The orifice must be inspected frequently, as it is likely to become clogged with suspended matter carried over from the mixing tank.

An example of a simple apparatus for temporary, small-scale use is shown in Fig. 218.[1] It was constructed by the Indiana State Board of Health from three barrels, a commercial constant-level regulating box, a small quantity of piping and a geared mixing contrivance much like that used on some types of ice-cream freezers.

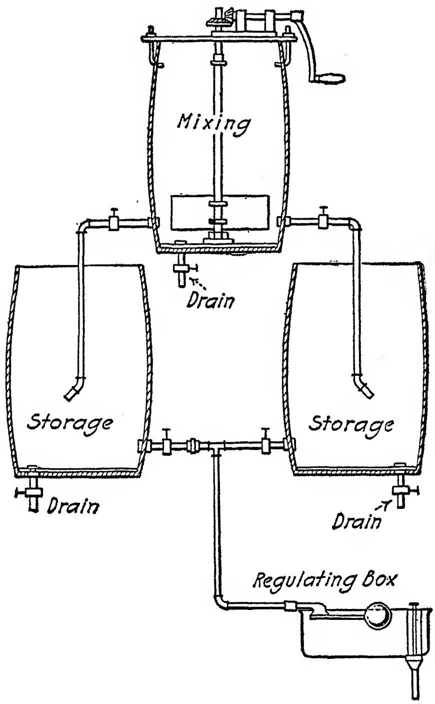

FIG. 218.—Emergency disinfecting apparatus, Indiana State Board of Health.

In practice, the preparation of a disinfecting solution from bleaching powder is disagreeable work on account of the dust and fumes, and more or less unsatisfactory because of the irregularity in the composition of the powder and consequent care needed to maintain a solution of proper strength, and the tendency of the measuring orifice to become clogged.

444. Chlorine Dosage.—As pointed out in Section 327, much of the chlorine added to sewage is used up by unstable sewage matters, before it can destroy the living organisms contained in

[1] *Engineering and Contracting*, 1912; **38**, 72.

the sewage. Chlorine demand may be exerted by certain kinds of mineral matter and by organic substances.

The oxidizable mineral substances are chiefly hydrogen sulphide, sulphites, nitrite nitrogen and ferrous iron. The important deodorizing effect of chlorine in sewage treatment is associated chiefly with its great affinity for H_2S, a foul-smelling gas. The quantity of chlorine used up by these mineral substances can be calculated readily from their combining weights. Thus 1 p.p.m. of H_2S will combine with $^{71}\!/_{34} = 2.1$ p.p.m. of Cl_2.

Since the constitution of the organic matter cannot be determined, its chlorine demand can only be found by test. Generally speaking, the chlorine demand of sewage parallels its oxygen demand.

For odor control a chlorine dose somewhat in excess of the amount theoretically required to combine with the hydrogen sulphide is required, because of the interfering demand of other sewage matters. For disinfection enough chlorine must be added to provide what is known as a "residual chlorine" content. State authorities have specified a disinfecting dosage varying from 20 to 25 p.p.m. for raw sewage to 15 to 20 p.p.m. for settled sewage and 5 to 15 p.p.m. for completely treated sewage. Since sewages and sewage effluents vary greatly in character it seems better practice to base the dosage on the residual chlorine to be secured. Values of 0.2 to 0.5 p.p.m. of residual chlorine after 10 to 15 min. contact appear to be adequate. The method of determining the residual chlorine should also be specified, as the starch-iodide test and orthotolidin test yield somewhat different results. Generally the latter is employed.

445. Efficiency of Disinfection.—The efficiency of disinfection depends upon (1) the quantity of chlorine added to the sewage and (2) the period of contact. Tiedeman[1] reports that at Huntington, N. Y., settled sewage could be disinfected satisfactorily when a residual of 0.2 p.p.m. after 15 min. by the orthotolidin test was obtained. From his studies he concluded that the Huntington effluent did not require a contact period in excess of 5 min. Contact periods of 15 min. or more are advised generally. While fine solids seem to be penetrated readily by chlorine, destruction of organisms imbedded in the coarser solids contained in raw sewage cannot be accomplished. The results

[1] *Eng. News.-Rec.*, 1927; **98**, 944.

obtained by Tiedeman at Huntington, N. Y., are shown in Fig. 219.

In order to exemplify the relative destruction of bacteria by chlorine, in comparison with their removal by other sewage

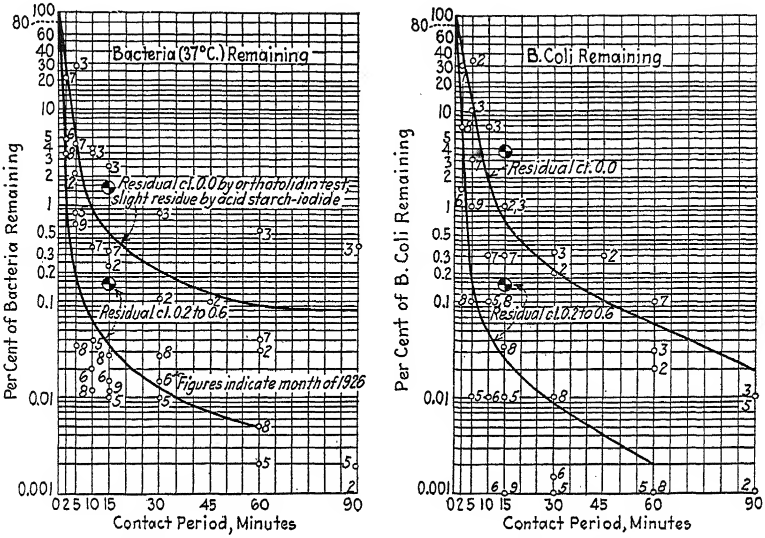

FIG. 219.—Efficiency of chlorination of settling tank effluent, Huntington, N. Y.

treatment methods, Table 108[1] is presented. It must be remembered, however, that the figures given are only approximations.

TABLE 108.—REMOVAL OR DESTRUCTION OF BACTERIA BY DIFFERENT TREATMENT PROCESSES

Process	Per cent removal	Process	Per cent removal
Coarse screens	0– 5	Trickling filters	90–95
Fine screens	10–20	Activated-sludge process	90–98
Grit chambers	10–25	Intermittent sand filters	95–98
Plain sedimentation	25–75	Chlorination of settled sewage	90–95
Septic tanks	25–75	Chlorination of oxidized sewage	98–99
Chemical precipitation	40–80		
Contact beds	80–85		

[1] ROSENAU: "Preventive Medicine and Hygiene," p. 1113.

446. Reduction of Biochemical Oxygen Demand by Chlorination.—The fact that much of the chlorine is used up by unstable sewage matters is reflected in a reduction of the biochemical oxygen demand of the sewage. A decrease in the 5-day, 20°C., demand of about 25 p.p.m. has been reported. This observation should probably be regarded as an oxidation of the readily oxidizable substances rather than as a delay of putrefaction. Reduction of the biochemical oxygen demand may be of importance in preventing nuisances below treatment works discharging their effluent into small streams which provide adequate dilution only by accession of other waters or by emptying into larger bodies of water.

DISPOSAL OF INDUSTRIAL WASTES[1]

447. The Problem of Industrial Wastes.—Stream pollution by industrial wastes has assumed increasing importance during the last fifty years, coincident with rapid industrial development. Conditions vary greatly, due to diversity of volume and quality of wastes and to differences in relative volumes of available diluting water.

The effect of many wastes upon waters into which they are discharged is similar to that of sewage, causing discoloration, turbidity, stimulation of fungal and algal growths, depletion of dissolved oxygen, extermination of fish life, putrefaction accompanied by objectionable odors, and deposits affecting navigation.

Unlike sewage, certain industrial wastes contain constituents sufficiently toxic directly to destroy fish life, while others, like acid mine waters, phenol wastes from gas plants and saline wastes from salt works and oil wells, render purification of waters suitable for domestic use difficult and in some cases impracticable. Most wastes, if discharged into bodies of water subsequently used for water supply, tend to increase the burden placed on plants for the purification of water for domestic uses.

Streams carrying tannery wastes may convey anthrax bacilli to large areas of grasslands, thus endangering cattle grazing thereon; relatively small quantities of glue factory wastes have caused excessive foaming of waters of large rivers, seriously interfering with their use for recreation; while waters have been rendered

[1] Adapted from Eddy, H. P.: "Disposal of Industrial Wastes in the United States," Trans., International Conference on Sanitary Engineering, 1924.

unsatisfactory for irrigation of crops by the wastes from mines and oil refineries.

There has been a tendency toward strict prohibition of the introduction of sewage and industrial wastes into streams without a high degree of prior purification. Such action tends to delay progress in securing reasonable control of pollution.

The magnitude of the problem of industrial wastes disposal is realized by few engineers, manufacturers and legislators. There are many single industries the polluting properties of whose wastes are equivalent to those of the domestic sewage from a city of 100,000 persons, and some of the wastes are far more expensive to treat than the sewage from an equivalent population. It is therefore of the utmost importance that stream pollution laws be administered with due regard for the "rule of reason," that treatment be regulated according to the actual needs in each case, and that treatment plants be designed and operated with a keen appreciation of cost on the one hand, and the accomplishment of the requisite results on the other.

448. Classification of Industrial Wastes.—Industrial wastes may be grouped into three main classes—animal, vegetable and mineral, according to the nature and origin of their predominant ingredients. This grouping indicates roughly the relative rapidity of putrefaction.

Some of the more common wastes are:

Wastes of animal origin, from
- Dairies
- Fertilizer industry (also mineral)
- Glue industry
- Leather-board industry
- Packing houses
- Soap industry (also vegetable)
- Tanneries
- Wool scouring plants
- Woolen textile mills

Wastes of vegetable origin, from
- Beet-sugar industry
- Canning industry
- Corn products industry
- Cotton textile mills
- Distilleries
- Paper industry
- Pulp industry
- Rubber industry

Saw mills
Straw-board industry
Wood distillation industry

Wastes of mineral origin, from
Bleacheries
Chemical industry
Dye works
Gas plants
Metal industry
Mines (coal mine drainage, coal washing, etc.)
Munitions factories
Oil refineries
Oil wells
Salt works
Water purification plants.

Wastes are often mixtures of materials of more than one class. For example, tannery wastes may contain animal fleshings, bark extractives and spent lime.

449. Effect of Industrial Wastes upon Municipal Sewage.— The influence of wastes upon the character of sewage depends upon two principal factors: (1) the relative volumes of wastes and sewage and (2) the composition of the wastes.

The great increase in weight of sewage constituents due to wastes has been illustrated in Section 269.

The increments of wastes such as those noted in Section 269 add greatly to the cost of treating sewage, by increasing the load upon the plants. Moreover, some wastes, such as acid pickling liquors, and copper and paint wastes, are detrimental to biological action and may necessitate modification in plants and processes otherwise suitable for sewage treatment.

Occasionally, however, wastes may be utilized to advantage, as formerly at Worcester, where the quantity of iron sulphate from pickling liquors was at times sufficient, with addition of lime, to cause excellent chemical precipitation.

Large quantities of heavy oils from industries and garages have formed troublesome scum in channels and tanks. In one town wool scouring liquors made treatment of the sewage upon intermittent filters impracticable, owing to clogging of the sand. At Atlanta carbide plant wastes caused a thick scum to form on the surface of the sedimentation compartments of Imhoff tanks.

Some cities have required partial treatment of wastes before their discharge into sewers, as at tanneries, where tanks and screens have been installed for removal of suspended solids; and

many cities have ordinances prohibiting the discharge of substances which will injure sewers and other structures or interfere with the treatment of sewage, but they have not always been enforced.

450. Treatment of Industrial Wastes.—Where industrial wastes must be disposed of separately, treatment of the wastes generally has been demanded to prevent:

Contamination of and esthetic damage to water supplies.

Damage to commercial fisheries, particularly contamination of shellfish.

Annoyance due to offensive odors.

Pollution of waters used for industrial processes.

Infection of cattle and pollution of their water supplies.

Pollution of streams to an extent which will interfere with their use for recreation—fishing, boating, bathing and camping.

The methods commonly used for the treatment of sewage are applicable to the treatment of many wastes. With others special adaptations of existing methods must be made, or new methods developed.

The volume of industrial wastes sometimes is increased unnecessarily by mixing relatively clean wastes resulting from one process with the concentrated wastes resulting from others. Separation of wastes for disposal of clean wastes without treatment and for treatment of objectionable wastes can often be effected with success. In other cases, mixing of different concentrated wastes will make them more easily amenable to treatment, and precipitating reactions frequently are observed. In still other cases the process waters can be used again and again until the waste products are sufficiently concentrated to render recovery or treatment economically attractive or practicable.

CHOICE OF SEWAGE TREATMENT METHODS

451. Governing Factors.—A number of different factors govern the choice of sewage treatment methods for any given municipality, sewerage district or sanitary district. Among them may be listed:

1. The character and volume variations of the sewage to be treated, taking into account the concentration, composition and condition of the sewage when it reaches the treatment and outfall works.
2. The required degree of treatment with respect to the ultimate disposal of the sewage by dilution or irrigation and the satisfactory disposal of the sewage sludge.

3. The practicability of obtaining sites for the treatment works and the conditions imposed by different sites, both as regards treatment methods and ultimate disposal of sewage and sludge.

4. The relative cost and efficiency of different treatment and disposal methods.

The important bearing of these factors upon sewage treatment has been touched upon in preceding chapters. It is well to point out at this time some of the intimate relations which exist between the treatment works and the remaining parts of the sewerage system.

To a certain extent, for example, the arrangement of the collecting system dictates the choice of the treatment methods and vice versa, because the treatment processes must be adapted to the requirements of the disposal site and to the facilities for ultimate disposal of sewage and sludge. The nature of the collecting system, furthermore, affects the composition, concentration and condition of the sewage delivered to the treatment works. Treatment methods vary for combined sewage and separate sewage, and for fresh and stale sewage. Different disposal sites may call for alternative plans of sewerage and sewage treatment. In a large city two or more treatment plants may be cheaper and better than a single one. Where land is expensive certain processes may become economical, while others are best suited where land is cheap. This is brought out in the following schedule prepared by one of the authors in a study of three different types of treatment works for a city of 600,000 persons.

	Area and volume of treatment plant	
	Acres	Million cu. ft.
1. Settling tanks, sludge beds and intermittent sand filters	800	140
2. Imhoff tanks, sludge beds and trickling filters	60	17
3. Activated-sludge process	10	5

In compiling figures for judging the relative merits of different methods of treatment, the cost of pumping, of a trunk sewer to convey sewage to the works, and of outfall sewers from the works must be considered a part of the total expense; sometimes these items are a large part of the entire amount. Interest, maintenance and sinking-fund charges on such structures must be combined with the fixed, operating and maintenance charges on treatment works in order to reach a true estimate of the annual cost of the method of treatment.

In preparing estimates of cost, the provision of land for extensions to meet future requirements must receive attention. This is particularly important where the treatment includes intermittent filters or other structures of large area in proportion to their capacity, and where it is probable that the degree of purification effected by the works must be increased with the lapse of time. Overworking treatment plants which it is impracticable to enlarge usually results in offensive conditions, and no plans for sewage treatment should be adopted which do not provide for a suitable increase of capacity during the period for which the works are designed. This period may be somewhere between 25 and 40 years, depending on local conditions. For example, it might be reasonably certain that clarification of sewage by screening and sedimentation, with disinfection of the effluent, will be a satisfactory treatment for 25 years, and enough land may be available near the city for works adequate for the requirements during that period, but no longer. Such a plant may prove more economical than the construction of a trunk sewer to a site much farther away but large enough for works of several times the utmost capacity of those nearer the city. It is true that the works close at hand may be serviceable for only a comparatively short period, but their total cost during that period may be less than the total cost during the same period of the works at a more distant site. It is hardly possible to estimate closely the future requirements of a city for more than 35 or 40 years, and in such a period of time sewage treatment may progress in many ways, so that it is unwise to plan works "for all time."

In all problems of sewage disposal, dilution as a means of disposal should be considered. The joint trunk sewers serving many of the cities and towns about Boston, Mass., and Newark, N. J., are examples of works constructed coöperatively, which relieve inland communities of treatment problems which would be serious in some cases. At Los Angeles, after an unsuccessful experience with sewage irrigation, it was considered best to construct an outfall sewer, 12.4 miles long, to the ocean. Seasonal changes in operation also must be considered, partial treatment in winter often sufficing, with complete treatment in summer.

452. Common Combinations of Treatment Processes.—With the many types of treatment processes available, it is only natural that the works of different communities should vary

appreciably in the combination of treatment processes employed. In order to acquaint the student with some of the most common ones the treatment methods used by a number of American municipalities are listed in Table 109.

TABLE 109.—TYPICAL SEWAGE TREATMENT PLANTS IN THE UNITED STATES

City	Sewage		Sludge	
	Treatment	Disposal	Treatment	Disposal
Bridgeport, Conn.	Fine screens	Dilution		
Dayton, O.	Racks, detritus tanks and Imhoff tanks	Dilution	Sludge-drying beds	To farmers, or used for filling
Marlborough, Mass.	Racks, plain sedimentation tanks and intermittent sand filters	Dilution	Sludge-drying beds	Used as fertilizer
Alliance, O.[1]	Racks, plain sedimentation and Imhoff tanks, contact beds and intermittent filters	Dilution	Sludge-drying beds	Used as fertilizer
Akron, O.	Racks, detritus tanks, Imhoff tanks, trickling filters and humus tanks	Dilution	Sludge-drying beds	To farmers or used for filling
Milwaukee, Wis.	Racks, grit chambers, fine screens, aeration tanks and sedimentation tanks	Dilution	Conditioning, Oliver filters and heat dryers	Sold as fertilizer

[1] This plant was replaced in 1929 by racks, grit chambers, Imhoff tanks, trickling filters and humus tanks.

453. Value of Experimentation.—The great advances in sewage treatment which have been made since the growth of cities and industries forced attention to this problem upon engineers, are associated, apart from laboratory research, with the studies at experimental plants of large communities. These were undertaken chiefly in order to select the most economical method of treatment under the given local conditions. Particularly noteworthy in this country are the studies made at Baltimore, Chicago, Cleveland, Milwaukee, New Haven, Philadelphia and Worcester. There can be no doubt that these studies paid for themselves, while advancing at the same time the art of sewage treatment and disposal. Sewage treatment remains a young art, and local conditions are so variable that new methods or combinations of old methods will continue to develop in the future.

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CHAPTER XXI

FINANCIAL CONSIDERATIONS

In every engineering project, financial considerations are of great importance. The engineer must be able to estimate the cost of work to be done, to compare the costs of alternate projects, and to give advice upon methods of raising money to meet the cost of construction and of operation of engineering works.

COST ESTIMATING

454. Definition.—Estimating involves the exercise of judgment in forming an opinion as to the value or quantities involved in operations concerning which the data are incomplete or inaccurate. In common usage, however, estimates include not only these approximate figures, in which the exercise of judgment is an important factor, but also precise arithmetical computations from known quantities and prices, as in "progress estimates" and "final estimates" for payments to contractors.

455. Kinds of Estimates.—Cost estimates are required for many purposes and are made in a variety of ways. Sometimes they are based largely or wholly on judgment, with little but the previous experience of the estimator to serve as a guide; again they are based on approximations as to the quantities and types of work to be done, with approximate prices for these items; at times quantities are accurately known while the prices must be approximated; and finally, both quantities and prices may be accurately known, and the "estimate" can be computed with precision.

456. Preliminary estimates are almost invariably required before the construction of any engineering work is undertaken. Two classes of preliminary estimates are recognized. The first is of a general nature in which the probable cost is estimated by comparison in a general way with the total costs of other works, adjusted merely for the relative size; for instance, the cost of a sewage treatment plant of a particular type to serve 100,000 persons may be approximated roughly from the

known cost of a similar plant designed to serve 125,000 persons. The second type of preliminary estimate is that prepared after drawings have been made in more or less detail for the work to be done. From such drawings the quantities of materials and of work of various classes can be computed satisfactorily and with a fair degree of accuracy; to these items are applied estimated unit prices based upon such pertinent data as the engineer can obtain, and the approximate total cost of each item of work is obtained by multiplying the estimated quantities by the estimated prices. Adding suitable allowances for other items of a general character not included in the items listed gives the total amount of the itemized preliminary estimate. Such an estimate is almost invariably required of the engineer in order to inform the client how much money he must raise for the work to be undertaken.

457. Contractors' preliminary estimates are made in a somewhat similar manner and serve as bases for the proposals submitted by bidders. The quantity under each item used in such an estimate is that given by the engineer or obtained by the contractor, who must estimate carefully the cost to him of doing each item of work, including proper allowances for contingencies, overhead expenses and profit.

458. Progress and final estimates, so called, are prepared by the engineer from time to time during the progress of construction work executed by contract and upon its completion. They are really computations of the sums of money due the contractor, and are based upon measurements of the quantity of work accomplished, to which the agreed prices are applied in accordance with the contract. The final estimate shows the amount of money to be paid to the contractor upon the completion of his work. It does not represent the total cost to the owner, as it does not include the cost of real estate or right-of-way, of materials furnished by the owner, of engineering services, and the like.

459. Estimates of value of existing works are occasionally required. Sometimes these are of a rough nature and are based, like general preliminary estimates, upon a mere comparison on the basis of size or of capacity with other works of which the cost is known. Sometimes they are itemized, the quantities being determined by measurements so far as practicable, with estimated prices applied to the items. In some cases, estimates of quantity

may be very precise where everything is accessible and can be accurately measured; in others, it may be difficult or impossible to determine accurately the quantities involved in parts of the work which are covered up or otherwise inaccessible.

Estimates of value prepared in this way generally are made on the basis of "reproduction costs." That is to say, the prices are those current at the time of the estimate, although the construction may have been done many years previous; and the resulting total represents approximately the cost of reproducing an identical structure. In order to obtain a fair figure for actual value it is necessary to make an allowance for depreciation, which is loss of value, the amount of which depends upon the kind of structure, its age, condition, and probable future term of service.

460. General Estimates.—Total costs of completed works may be reduced to unit costs on certain general bases such as capacity, population served, area, or other appropriate unit. Within reasonable limits such unit costs, particularly if derived from a number of plants recently constructed, may serve as a basis of estimating roughly the total cost of similar projects.

461. Unit Prices.—The determination of the appropriate prices to use in estimating the cost of any proposed construction is difficult. The most common method is by the application of unit costs for structures actually completed, particularly if they have been done in the same general locality and in recent times. Figures in cost data handbooks may sometimes be of service, but many times are unsatisfactory because the work was done years ago, or in a distant locality, or because the descriptions of the work are incomplete and make it impossible to judge whether all items of cost are included. Trade journals and professional periodicals occasionally contain articles describing construction work with cost details, and comparisons of the bids received on large contracts are sometimes given. These data are useful if carefully analyzed, and if sufficient information is available as to local conditions and the items included in the work to be done. Differences in methods of grouping items may make such figures misleading unless the user is thoroughly familiar with the details. For instance, an item which is on the bid sheet as "pipe laying" may include excavation and backfill of the trench, with the laying and jointing of pipe furnished by the owner; in another case it may include the same items of

work and also furnishing the pipe; in still another case it may include merely handling and jointing the pipe, while excavation

METCALF & EDDY ENGINEERS BOSTON, MASS

DAILY FORCE ACCOUNT AND TIME SHEET

JOB Boston Contract 3 INSPECTOR John Richard DATE Apr. 19, 1929

	Labor or Materials	Rate	Hours	Total Hrs	Labor Cost	Equipment Type & Hrs	Equipment Rate & Cost	Quantities	Notes
Overhead	Supt.	1.50	1 1	9	13.50				
	Timekeeper	.50	1 1	9	4.50				
	Water Boy	.25	1 1	9	2.25				
	Watchman	5.00			5.00				
Earth Excavation	Foreman	1.00	1	5	5.00	Marion		From Sta. 3+25	Street
	Shovel Oper.	1.25	1 1	9	11.25	Shovel		To Sta. 4+40	Beacon
	Laborers	.70	3 3	27	18.90	1/2 C.Y.		Width 3'	Type of Soil
							30.00	Depth 7'	Gravel
								C.Y. 89.5	
								Cost per C.Y.	
Rock Excavation	Foreman	1.00	1	2	2.00	Ingersoll		From Sta. 3+00	Street
	Compressor Op.					Rand		To Sta. 3+50	Beacon
	Drillmen	.80	4 4	36	28.80	Compressor		Width 3'	
	Laborers							Depth 3'	
								C.Y. 17'	
							12.00	Cost per C.Y.	
Sheeting	Foreman							From Sta.	Tight or Skeleton Sheeting
	Carpenters							To Sta.	
	Laborers							Depth to Bottom	Size of Rangers
	Materials							Spacing	
								Sq. Ft. Bank	
								Cost per Sq. Ft.	
Pumping	Pump. Oper.								
	Laborers								
Backfilling	Foreman	1.00	1	2	2.00	Tractor		From Sta. 2+10	
	Operator	.75	1 1	9	6.75	Plow		To Sta. 2+90	
	Laborers	.70	3 3	27	18.90			C.Y. 71	
								Cost per C.Y.	
							10.00	Cost per L. ft.	
Pipe Laying	Foreman							From Sta. 2+20	Type
	Pipe Layers	.80	2 2	18	14.40			To Sta. 3+00	Vit.
	Laborers	.70	2 2	18	12.60			Length 80'	Kind of Joints
								Size 15"	G.K.
	Materials		80 ft. - 15" pipe					Cost per lin. ft.	
Concrete Work	Foreman							From Sta.	Mix
	Mixer Oper.							To Sta.	
	Carpenters							C.Y. per lin. ft.	
	Laborers							Total C.Y.	
								Cost per C.Y.	
								Cost per lin. ft.	
	Materials							Cost of Forms	
Manholes Catch Basins	Foreman							Sta.	Street
	Masons							Depth to invert	
	Masons Help							No. Bricks	
	Carpenters							Bags Cement	
	Laborers							Bags Lime	
								Steps	
								C.Y.	
								Cost lin. ft.	
	Materials							Cost C.Y.	
Paving	Pavers							From Sta.	Street
	Laborers							To Sta.	
								Width	Type
								S.Y.	
	Materials							Cost per S.Y.	
									Weather
									Fair

FIG. 220.—Daily force account sheet.

and backfill may be paid for by the linear foot or the cubic yard under another item.

Fig. 220 shows a daily force account sheet which has been used by the authors for securing data upon cost of construction work. This form has proved satisfactory, the data obtained in this way being analyzed later in the office.

462. Fluctuations in Cost (Index Numbers).—Changes in basic conditions, particularly those caused by the war, have resulted in considerable changes of unit costs as compared with those prevalent in earlier years. Curves showing the fluctuation of index numbers, representing average current costs of certain commodities which are assumed to be typical, have been prepared by various agencies with the object of making it possible to reduce cost figures to a uniform basis of comparison, or to utilize the cost figures of earlier years by the application of suitable factors to make them comparable with current costs. Rates for labor have increased considerably, particularly during the war, but also to a lesser degree since the war. Prices for most commodities increased markedly during the war, and some have continued to increase while others have decreased since that time. The developments in the use of machinery, however, have been very great, resulting in increased production, and have had an important effect upon unit costs of engineering construction.

Fig. 221 represents a typical fluctuation curve of the index numbers published monthly in Engineering News-Record, based on prices of cement, steel, lumber and labor. These index numbers are more properly applicable to building costs than to general engineering construction, although they have been used to a considerable extent for the latter purpose. Curves showing the fluctuation of cost of cement, vitrified pipe and common labor have been added to the diagram.

The engineer may obtain original cost data from records kept by his own field force or from the contractor, if he keeps adequate records. While some contractors refuse to furnish such data, others generously coöperate with the engineer, many times to the advantage of both.

The best means available to the engineer for obtaining actual cost data are through the efforts of his resident engineers and inspectors, working in coöperation with the contractor. The labor rates, cost of equipment, supplies and materials, and overhead expense often may be obtained from the contractor, but if he prefers not to furnish such information, it can usually be estimated with moderate accuracy from such information

as the resident engineer can obtain. Intimate acquaintance with conditions under which the work is done makes the data obtained by the engineer of much more value to him than data derived from any other source. A fund of such data accumulated

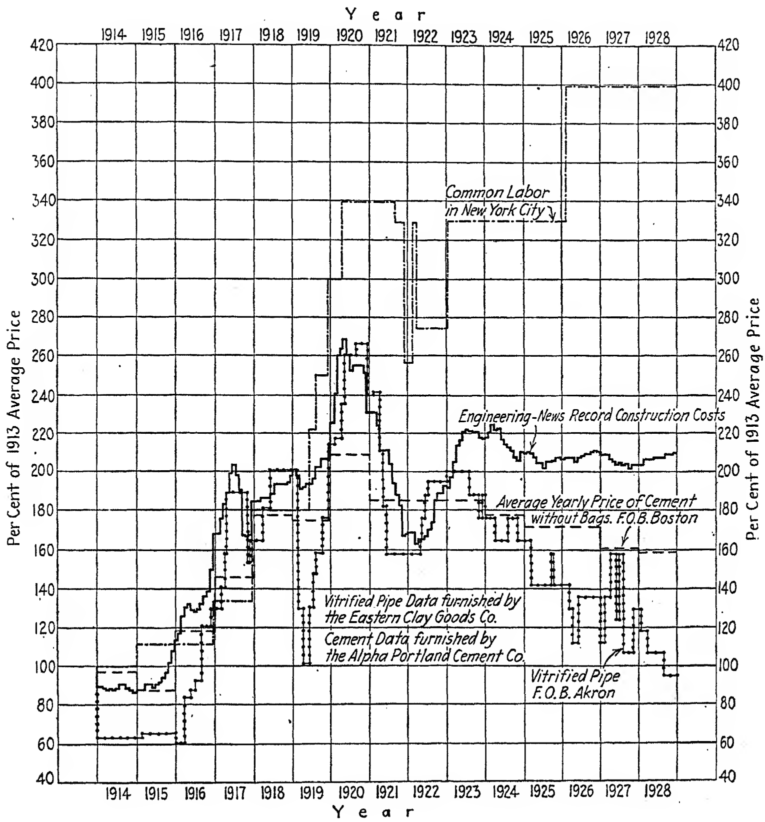

FIG. 221.—Curves showing fluctuations of construction costs, wages of common labor in New York City, prices of cement and of vitrified clay sewer pipe. Curves based on average prices in 1913 = 100 per cent.

over many jobs will prove of substantial value to him in preparing estimates.

In compiling cost data it is necessary to remember that results obtained for a day or other short period of time cannot be taken as representative. The output of men and machinery fluctuates from day to day for many reasons, and only the average for the whole work should be considered in estimating other work.

If it be assumed that the Eng. News-Rec. index numbers are applicable to the type of construction under consideration, an estimator having cost data obtained in 1914 and wishing to use them in connection with work planned in August, 1928, would proceed as follows: the index number for 1914 was 88, and for August, 1928, it was 207.29; then, if the unit cost in 1914 was \$1.50, the corresponding price in August, 1928, would be \$1.50 $\times$ 207.29/88, or \$3.54.

Such curves of index numbers should be used very cautiously if at all. Their indiscriminate use may give misleading results. This is particularly the case with regard to cost of excavation done by machinery, which bears little relation to cost of labor or of such commodities as are utilized in making up index numbers.

The index number properly applicable to a large undertaking, the construction of which extended over a period of several years and which may have been handled in several contracts, must be chosen with caution. The selection of a number with reference to a single date, such as that of the beginning of the work, is likely to be erroneous, since some large contracts may have been let a year or two later at a time when conditions differed materially from those at the outset. A weighted average value of the index number may be obtained, if the date of letting and the contract price of each of the major contracts are known, by multiplying each contract price by the index number corresponding to the period just preceding the receipt of bids, adding the results together and dividing by the sum of the contract prices.

Knowing the cost of a particular kind of work in a given locality, the wages of labor in that locality and the proportion of the total cost applicable to labor, it is possible to approximate roughly the cost of similar work elsewhere and with the labor rates different by pro-rating the labor portion of the cost according to the change in labor rates. If considerable time has elapsed, however, involving changes in methods and the use of machinery, such estimates are likely to be inaccurate.

463. Synthetic estimates of unit prices sometimes may be prepared, and if the data are sufficient for their proper compilation, they may be more trustworthy than those derived from costs of other work. In particular, contractors' estimates of probable cost often are made up in this way, combining the costs of labor, materials and equipment in accordance with the

circumstances. Few engineers have sufficient data of a character to enable them to prepare such estimates with accuracy, and unless founded upon reliable data or mature experience figures of this kind may be distinctly misleading. It is particularly important in preparing such estimates to have accurate records of cost of plant and output of each unit, for with the development and rapid progress in the use of machinery, unit costs are being affected by it more than by any other factor.

464. Overhead.—In addition to the costs of the several items involved in a construction project, there will be other substantial costs which have no more relation to one part of the work than to another, but are necessarily met in connection with the job as a whole. These costs are classed as "overhead," and may include the wages of superintendent, timekeeper, clerks, watchmen and other employees whose time cannot be allocated to definite parts of the work; the payments for insurance, surety bonds, field office, telephone, light, heat, water, permits and licenses, small tools and equipment, and sometimes many other items. There is no uniformity of practice regarding items included in overhead. On a large piece of work, overhead costs may be many thousands of dollars; on a small job where the contractor acts as his own superintendent and does not require an office they may be very small, but may well constitute a larger proportion of the total cost of the job than in the case of the larger undertaking.

It is customary in engineers' detailed estimates to price the items at direct cost, as nearly as possible, without allowance for overhead and contractors' profit. Allowances for these items may then be made by adding a percentage for each or covering both in one computation. In a contractor's estimate made as a basis for bidding on a unit price contract, the allowances for overhead and profit must be distributed to the individual items; otherwise an increase or decrease in the quantities will not correctly affect these allowances.

465. Contractors' Profit.—It is obvious that a contractor cannot continue in business unless he makes a profit. A reasonable profit on all work best serves the interest of both the owner and the contractor. Proper allowance for such profit should therefore be made in all estimates. It may be and frequently is considered in connection with overhead, and a single percentage taken to cover both items.

466. Contingencies.—It is rarely the case that everything to be done in carrying out construction work of considerable magnitude has been foreseen and included in the estimates, and there is also a real possibility that quantities may over-run those estimated. Moreover, various causes may make the cost of doing certain work greater than was anticipated. The result will be that the total cost of the work will exceed the sums of the costs estimated by items. The various causes which may produce such increased cost are classed as contingencies.

The engineer must consider contingencies as likely to involve items not included in his estimate, and also to increase the quantities of the items listed; it is possible too that conditions encountered in the work may demonstrate that his unit prices were too low. He must provide for these possibilities by adding an allowance for contingencies.

The contractor is not so much concerned with possible omissions or increases in quantities if the contract is on a unit-price basis, since he will be paid in proportion to the actual quantity for each item; but in case of a lump-sum contract the possibility of additional items or increased quantities is of importance to him. Whatever the type of contract, he must give particular consideration to matters which may increase the cost of his work, such as storms, fires, accidents, greater difficulties than were anticipated, delays, increase in prices of materials to be used, and the like; and the only way in which he can meet such conditions is by suitably increasing the unit prices for the several items in his estimate.

The allowance for contingencies should be varied in accordance with the degree of uncertainty involved. In a preliminary estimate, where comparatively little has been done in the way of investigating possible difficulties, an allowance of as much as 20 per cent may be appropriate, whereas in cases where the plans have been matured after thorough study and detailed investigation, including adequate exploration of underground conditions when such are involved, the allowance may be reduced to 10 per cent or less.

467. Engineering Costs.—Allowance for the cost of engineering frequently is combined with that for contingencies, in estimates of cost, although they are obviously distinct items. It is a fact, however, that the more liberal the allowance for engineering, the smaller will be the cost of contingencies; and therefore

the percentage allowance for "contingencies and engineering" may logically vary less than that for either of the items taken singly. The provision of funds for engineering is sometimes inadequate to permit the thorough studies required for first-class engineering. Under such circumstances the cost of the completed work, whether measured in first cost or in value of the structure, may greatly exceed the additional cost of the engineering which would have been involved if more thorough studies had been made.

A common allowance for "contingencies and engineering" is 15 per cent. This may be adequate in cases where the study is sufficiently detailed to justify the allowance of but a small amount for contingencies, and where the magnitude of the work is such that the engineering cost alone will not involve a comparatively high percentage; but in many cases, particularly on work of moderate size, an allowance of 20 per cent would be better. On rough preliminary estimates it may be wise to allow as much as 25 per cent.

468. Interest during Construction.—The interest upon money invested in the construction before its completion is an item of cost of construction, and in a large enterprise requiring several years to build this may amount to a considerable sum. For instance, if 3 years elapse from the beginning of the enterprise to its completion, and the money is paid at a uniform rate throughout that time, the cost of interest during construction will be one and one-half times the annual interest on the entire cost.

469. Illustrative Example.—What will be the cost of substructure for a small pumping station to be built by contract, with data as given for each type of estimate?

(*A*) Preliminary general estimate in 1928.

Data.—Cost of a similar structure under comparable conditions in 1923 was \$9,400. Index number in 1923, 220; in 1928, 207. Estimated cost in 1928, $^{207}/_{220} \times \$9,400 = \$8,850$.

Note that such an estimate assumes that the index figures represent fairly the relative effect on cost of machinery and equipment, which is somewhat inaccurate.

(*B*) Preliminary itemized estimate of contract cost, made by engineer after completing his plans and quantity surveys, which will be used as the basis of comparing bids.

Item	Quantity	Unit price	Amount
Excavation, including pumping water.	300 cu. yd.	$10.00	$3,000
Backfilling........................	60 cu. yd.	0.50	30
Concrete........................	133 cu. yd.	20.00	2,660
Reinforcing steel.................	16,000 lb.	0.05	800
Damp-proofing.....................	2,000 sq. ft.	0.05	100
Sluice gates and stands.............	....	......	300
Cast iron pipe and fittings...........	....	......	300
Miscellaneous steel and iron.........	....	......	150
			$7,340
Contractor's overhead and profit, 20 per cent.....................	....	......	1,468
Estimated amount of contract........	....	......	$8,808
Contingencies, 10 per cent...........	....	......	881
Estimated total cost of contract......	....	......	$9,689

(*C*) Contractor's bid, being his estimate of cost with overhead and profit distributed to the several items.

Item	Quantity	Unit price	Amount
Excavation, including pumping water.	300 cu. yd.	$12.00	$3,600
Backfilling........................	60 cu. yd.	0.60	36
Concrete........................	133 cu. yd.	24.00	3,192
Reinforcing steel.................	16,000 lb.	0.06	960
Damp-proofing.....................	2,000 sq. ft.	0.065	130
Sluice gates and stands.............	....	......	350
Cast iron pipe and fittings...........	....	......	360
Miscellaneous iron and steel.........	....	......	125
			$8,753

Compared with engineer's estimate of $8,808.

(*D*) Synthetic estimate upon which contractor based his unit price of $12.00 for excavation.

Crane rental and operating supplies, 2 weeks at $120	$ 240
Transportation of crane	50
Compressor, 2 weeks at $60	120
Sheeting hammer, 2 weeks at $20	40
Pump, 2 weeks at $40	80
Truck, 6 days at $25	150
Steel sheet piling, 30 tons at $55 = $1,650, less salvage, $400	1,250
Lumber for bracing, 1.5M at $60	90
Foreman, 2 weeks at $75	150
Crane operator, 2 weeks at $75	150
Carpenter and mechanic, 24 days at $12	288
Laborers, 72 days at $5	360
Small tools and miscellaneous items, including fuel	75
	$3,043
Contingencies, overhead and profit, 20 per cent	609
	$3,652

$3,652 ÷ 300 cu. yd. = $12.17, say $12.00.

(*E*) Final estimate for payment to contractor.

Item	Quantity	Unit price	Amount
Excavation, including pumping water	312.4 cu. yd.	$12.00	$3,748.80
Backfilling	72.0 cu. yd.	0.60	43.20
Concrete	141.2 cu. yd.	24.00	3,388.80
Reinforcing steel	15,870 lb.	0.06	952.20
Damp-proofing	1,948 sq. ft.	0.065	126.62
Sluice gates and stands	...	...	350.00
Cast iron pipes and fittings	...	...	360.00
Miscellaneous iron and steel	...	...	150.00
Extra work (would be shown in detail)			
Total cost		$74.88	
15 per cent		11.23	86.11
Total			$9,205.73

Note that prior payments to the contractor, if any, made upon the basis of progress estimates, must be deducted to show the amount payable under the final estimate.

COMPARING COSTS OF ALTERNATE PROJECTS

470. Construction Costs.—The preceding sections of this chapter relate to the costs of construction of engineering works. Comparison of estimated construction costs of alternate projects frequently is made, but does not constitute a fair comparison,

unless the costs of maintenance and operation are the same for both projects, and their useful lives identical. If works may be expected to last forever, and to involve no costs of operation and maintenance, then construction costs (including cost of land required) may be compared with advantages.

471. Maintenance and Operation Costs.—The work of operating and of maintaining sewerage works generally is done by one organization, and while there is a distinction between operation and maintenance, the importance of keeping the accounts separated is not sufficient to justify the care required on the part of the accountant to divide the labor into separate accounts. For this reason these accounts usually are combined. Operating costs have to do with running the plant; maintenance costs are for keeping the works in running order.

472. Total Annual Charges.—The annual charges include the cost of maintenance and operation; interest upon the construction cost; and a replacement item sufficient to replace, at the end of their useful life, those parts of the plant which wear out or become useless.

Interest.—From the point of view of economics, it is immaterial whether a capital expenditure is made out of funds on hand, thus losing the interest upon that sum, or whether the money is borrowed for all time, requiring an annual payment of interest in perpetuity. The annual cost to the owner of the plant for interest is the same in either case, if the rates of interest are identical. It is not good public policy, however, to plan for interest payments over a very long time, say in excess of 20 to 40 years, for sewerage works of the several kinds.

Replacement Fund.—If a plant wears out in a definite term of years, and must be replaced at the same cost as that of the original plant, while the new one will wear out in the same number of years, then a sum can be set aside annually which, put at interest, at the end of the useful life always will amount to the cost of the plant to be replaced. The annual replacement fund is this amount which must be contributed annually to a sinking fund to provide for the renewal of the plant at the end of its life. If there were no changes in cost of construction or rate of interest, and if the identical plant were to be replaced each time it wore out, the annual contribution to the replacement fund would be the same for all time; these assumptions will seldom if ever be correct, but provide a basis for comparisons. This cost of replacement is

sometimes figured as equivalent to an annual depreciation of a given percentage. Assuming the life of the part of the plant under consideration as 20 years, the depreciation might be taken at a uniform rate of 5 per cent per annum. This is an easier, though less logical, method for determining the replacement cost. It neglects interest earned upon the sums set aside for depreciation, and is not to be recommended, although it may be sufficiently near for rough comparisons.

473. Comparison of Annual Charges.—The foregoing are all the items of theoretical annual cost; and if there is no change in costs or interest rates and no larger or different plant becomes necessary, the perpetual payment of the total annual charges will take care of the plant and of the operating and maintenance costs. Such annual charges may therefore be compared directly, and the project involving the lowest annual cost will be economically the most advantageous, if the assumptions made are justified.

As a matter of fact, however, construction costs do vary materially, and the cost of replacement may be greater or less than that of the original plant; interest rates also change; and there is frequently a difference between the rate of interest to be paid upon borrowed money and that which a sinking fund can earn. Moreover, it is impossible to estimate exactly in advance the useful life of a plant, and when it requires renewal it is likely that it will also require enlargement, while possibly an entirely different type of plant may be advantageous. Operating conditions and consequent costs of operation and maintenance do not remain constant. For these reasons it is not possible to make a true comparison of relative economic advantages of various projects, but comparisons on the basis described are of value as guides to judgment.

474. Capitalized Costs.—The capitalized value of any constantly recurring cost is the sum of money which, put at interest, will produce the income necessary to pay such recurring cost. Thus, if the operation of a plant costs \$400 per year, and the interest rate be 4 per cent, the capitalized cost of operation is $\$400 \div 0.04 = \$10{,}000$.

Comparisons of alternate projects are made more frequently upon the basis of capitalized costs than of annual charges. If all items are included, the results will be the same by both methods. Obviously the capitalized value of the interest upon cost is the cost itself. Consequently the sum of the construction cost and

the capitalized values of the cost of operation and maintenance and of the contribution to replacement fund provides the proper figure for comparison.

These comparisons are helpful, but not conclusive. It is not good public policy to spread payments of principal and interest over a longer term of years than is necessary, even though it may be theoretically advantageous.

475. Illustrative Examples.—A typical example illustrating this method of comparing costs involved in two alternate projects may demonstrate the method of procedure. A trunk sewer carrying a flow of 750,000 gallons per day is to be built. The sewer must either go through a hill, requiring a tunnel 1,000 ft. long in rock, or be built over the hill in a trench at ordinary depth, in which case it will be necessary to pump the sewage to a height of 75 ft. and then let it flow down the hill by gravity. The latter scheme involves the provision of a pump house with screens, two pumps and a pipe 100 ft. longer than the tunnel. Estimated costs for the two projects are as follows:

Construction cost of gravity sewer through tunnel..	$20,000
Construction cost of works including pump house, two pumps, screens and pipe sewers.............	12,000

The high level sewer would be the cheaper in first cost, but if the cost of operation, interest and replacement are taken into account, the following comparison results:

	Tunnel	High level sewer
Operating and maintenance expenses:		
Power, supplies, repairs, attendance..........	$100	$2,500
Interest on construction cost at 4 per cent......	800	480
Replacement fund............................	200	300
Total annual cost............................	$1,100	$3,280
Capitalized at 4 per cent....................	$27,500	$82,000

This would indicate that it would be cheaper in the long run to build the tunnel, although the first cost is much higher.

476. Data for Use in Problems.—Table 110 contains figures of approximate unit costs of construction and operation of two types of sewage treatment plant, which may be used in the solution of problems in this chapter. No attempt has been made to go into detail, and obviously actual costs would vary

considerably with the size of the plant as well as with local conditions. The figures should be taken as illustrative rather than authoritative statements of actual costs.

TABLE 110.—APPROXIMATE UNIT COSTS OF CONSTRUCTION AND OPERATION OF SEWAGE TREATMENT PLANTS AND ALLOWANCES FOR REPLACEMENT; FOR USE IN SOLUTION OF PROBLEMS

Type of plant	Cost of construction		Cost of operation		Annual replacement allowance, per cent of construction cost
	Per capita to be served in future	Per million gallons daily capacity	Per capita actually served per year	Per million gallons treated	
Tanks and trickling filters	$13.00	$120,000	$0.30	$7.60	2.401
Activated sludge.........	11.00	101,500	1.00	25.30	1.783

The figures in this table, while providing a basis for solving illustrative problems, should not be taken as typical of actual costs in any particular locality.

METHODS OF FINANCING

Municipalities have four methods of raising money for the construction and operation of sewerage works, *viz.*, by issuing bonds, by general taxation, by special assessments, and by charges for service.

Still another method by which sewer construction is sometimes financed to a limited extent is through private enterprise, no public funds being used.

477. Bonds.—The construction of sewerage works of considerable magnitude usually involves a cost which cannot be met out of the taxation or other municipal revenues of a single year, and it is therefore necessary to borrow money on bonds in order to provide the necessary funds.

Bonds are promises to pay definite sums of money at stipulated dates, together with interest (generally payable semiannually) at a fixed rate. Three types of bonds have been used in municipal financing: those containing no definite provision for payment, and therefore likely to involve the issuance of a new set of bonds to refund the debt upon the maturity of the original bonds; those providing for the establishment of a sinking fund[1]

[1] A sinking fund is usually built up by annual installments, with their accumulations of interest, compounded. If S is the amount of the debt, I the annual installment, r the rate of interest (written as a decimal) and n the term in years, and if interest is compounded annually, then

$$I = \frac{Sr}{(1 + r)^n - 1}$$

designed to accumulate sufficient funds to pay the bonds when due; and those providing for serial payments, so that some of the bonds mature and are paid each year until the whole issue is retired. Table 111 contains figures of sinking fund contributions required for several periods and rates of interest.

The term of years for which municipal bonds may be issued is regulated by law in many states. It should never exceed the useful life of the works to be built with the proceeds of the bond issue. It is not considered good public policy to issue bonds for longer terms than 40 years, even though the works to be built may last for centuries.

478. Relative Economy of Various Bonds.—As a matter of economics, there is no difference in the relative economy of bonds of the several types, and of different terms of years, if a single rate of interest is applicable to all; nor is there any difference between paying cash in full and borrowing in perpetuity. However, the interest rate on sinking fund deposits is frequently less than that paid on the bonds, and this difference may affect the relative costs.

The true basis for comparison is the present worth of all the sums to be paid under any plan of financing. For instance, if the rate of interest be 4 per cent, there is no economic difference between paying out $1,000 in cash or paying $40 per year for all time.

TABLE 111.—DATA FOR COMPUTING SINKING FUNDS AND REPLACEMENT CHARGES, GIVING YEARLY PAYMENTS REQUIRED TO REDEEM $100 AT END OF ANY YEAR

Years	Rate of interest, per cent				
	3	3½	4	4½	5
5	18.84	18.65	18.46	18.28	18.10
10	8.72	8.52	8.33	8.14	7.95
15	5.38	5.18	4.99	4.81	4.63
20	3.72	3.54	3.36	3.19	3.02
25	2.74	2.57	2.40	2.24	2.10
30	2.10	1.94	1.78	1.64	1.51
35	1.65	1.50	1.36	1.23	1.11
40	1.33	1.18	1.05	0.93	0.83
45	1.08	0.95	0.83	0.72	0.63
50	0.89	0.76	0.66	0.56	0.48

Annual payments are made at the end of each year, and interest is compounded annually.

Frequently it is argued that the total amount of money paid out in interest and capital payments is the significant matter, and attention is called to the fact that a \$100,000 serial bond issue at 4 per cent will involve total payments of \$122,000 if it runs for 10 years; \$142,000 if for 20 years; and \$202,000 if for 50 years. Similar payments for a sinking fund bond (sinking fund drawing 3½ per cent) will be \$125,241, \$150,722 and \$207,309.[1] While these statements are correct, they take no account of the present worth[2] of future payments, and the comparison is somewhat misleading.

There are certain advantages and disadvantages in the use of each type of bond. Experience indicates that officials have sometimes mismanaged or misused sinking funds, and in at least one state (Massachusetts) sinking fund bonds cannot now be used for financing public improvements. They have the advantage of requiring a uniform payment throughout the life of the bond, while serial bonds require a high initial payment, decreasing as the bonds are retired. Since the relative ability of the municipality to pay is just the reverse of this, the serial bond imposes an unbalanced burden upon the community. However, as a matter of public policy there is serious objection to long terms for bonds, and to the continuance of interest payments for many years.

479. General Taxation.—It is usually impracticable to raise by general taxation in a single year sufficient funds for the construction of major sewerage works, although the cost of minor extensions may be included in a tax levy. The money required for important works must usually, as already noted, be raised in the first instance by an issue of bonds. The money for paying interest on such bonds and the contributions to sinking funds or payments of bonds maturing serially must usually be obtained partly or wholly from the proceeds of general taxation. Oftentimes the money required for operation and maintenance must also be included in the tax levy.

Unless provision is made for raising at least a part of the funds required annually through special assessments or charges for service, or both, the requirements must be met by general

[1] *Eng. News-Rec.*, 1917; **79**, 407.

[2] The present worth P of a sum A to be paid at the end of n years is

$$P = \frac{A}{(1 + r)^n},$$

r being the rate of interest which is to be compounded annually.

taxation. It is a common provision of law that, unless funds for bond interest and sinking funds or maturing bonds are provided from some other source, assessors *shall* include them in making up the tax levy.

480. Special Assessments.—In many cases special assessments are made against property especially benefited by the construction of a sewer, to cover a part or all of the cost. Such special or benefit assessments can only be levied when the property so taxed is increased in value or otherwise benefited above and in addition to the general benefit accruing to the entire community, and the amount of the special assessment must not exceed the value of the benefit obtained.

In the case of lateral sewers, a considerable proportion of the cost, often from one-third to three-fourths, is adopted as a proper benefit to be assessed against abutting property, the remainder being assumed by the community as a whole. In some cases an additional amount is assessed to include a small proportion of the cost of main sewers and of treatment works, since the lateral sewer could not actually benefit property but for the main sewers and perhaps the treatment works, which convey away and dispose of the sewage.

Special assessments are frequently payable in installments over a term of years, commonly 3 or 5 years, and are usually collected in the same way as general taxes.

Several methods of computing the assessments have been used; the most common are based upon frontage, upon area, or upon a combination of frontage and area.

Fixing the assessments upon the basis of frontage is the simplest to apply, but unless lots are of the same shape and size this may be distinctly inequitable. It is conceivable that an abuttor might have a very small frontage upon the street, but maintain a large building upon a rear lot; and if he paid only on the basis of frontage would pay much less than another owning a shallow lot with considerable frontage. Moreover, a corner lot might be assessed upon frontage on two streets, although the property might be fully served from one street. The latter objection is frequently met by exempting a certain frontage, such as 60 to 100 ft., from assessment on a second street.

Assessments based upon area alone are also likely to be inequitable, especially if lots are irregular. In any event, land lying more than 100 or 125 ft. back from the street may be benefited

little if any by the presence of the sewer, and such distant land is usually exempt from such assessment.

In basing assessments upon a combination of the frontage and area, the total cost which is to be assessed is divided into parts, as one-third to be assessed upon frontage and two-thirds to be assessed upon area. The net assessable frontage, after deducting exemptions, divided into the amount to be assessed upon frontage gives the assessment rate per linear foot; and the net assessable area divided into the amount to be assessed upon area gives the assessment rate per square foot. The sum of the frontage and area assessments gives the total assessment for each lot.

In some cases the problem of equitably distributing special assessments is complicated. Where a district under development includes much farming land not yet subdivided but nevertheless benefited by a sewer, it may be possible to compute the present worth of a future benefit so as to assess such farming land its reasonable proportion of the cost.[1]

481. Charges for Service.—Until recent years, charges for sewer service have not been common, but there is now (1929) a distinct tendency to make such charges. There can be no doubt of the propriety of annual charges for the use of sewers, sufficient at least to pay the costs of maintenance and operation, including the costs of treatment and disposal of the sewage.

Where such charges are made they may well include a sum for "readiness to serve," which may be based upon the number and kind of fixtures connected; and another for use of the system, based upon the quantity of sewage discharged, as nearly as it is practicable to ascertain it. In general, the quantity may be taken as equivalent to the volume of water consumed.

Since the payment of charges for sewer service cannot be enforced by cutting off the sewer connection, it is important that there be laws, regulations, or contracts enabling the municipality to collect bills for such service. Where state laws permit making such charges liens upon property, the collections may be made in the same way as taxes. In other cases it may be possible to collect bills for sewer service with those for water, and to enforce their payment by cutting off the water service if necessary.

[1] Bradbury: "County Sewer District Work in Ohio and Assessments of Cost According to Benefits," *Proc.*, A. S. C. E., 1928; **54**, 2061.

482. Private Financing.—It may sometimes be impossible to induce a municipality to construct sewers in newly developed streets, and in such cases the sewers may be constructed at the cost of the owner of the property. Sometimes such sewers are immediately donated to the municipality, the owners considering the cost as representing in effect a benefit assessment upon their land. Sometimes they remain private sewers for a time, but are later purchased by the municipality, or taken when the streets are accepted as public ways.

When sewers which are to discharge into the public sewers and will eventually form part of the municipal system are constructed by private enterprise, they should be laid out by or subject to the approval of the city authorities, and should be constructed under the same specifications as sewers built by the city.

Problems

1. Estimate the cost of a 12-in. vitrified pipe sewer in a city street if the average depth to invert is 10 ft.; length of sewer, 575 ft.; two manholes required; width of trench, 3 ft.

Unit prices: excavation and backfill, $1.50 per cubic yard; 12-in. vitrified pipe laid, $1.00 per linear foot; manholes, $15.00 per foot of height above invert, including castings; paving, $1.75 per square yard. Allow 15 per cent for overhead and profit, and 10 per cent for contingencies and engineering.

2. The solid lines on Fig. 222 show a pipe sewer as originally laid out. During construction a pipe was discovered at Sta. 0 + 95, which could not be disturbed, and a new layout to clear the obstacle had to be made as

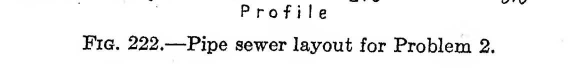

FIG. 222.—Pipe sewer layout for Problem 2.

shown by dotted lines. A 15-in. pipe was required to replace the 12-in. sewer in this stretch. Using the same prices as given in Problem 1, what is the estimated cost of the sewers on original and revised locations? No intermediate manhole was needed for the original line. Width of trench, 3 ft. Cost of 15-in. vitrified pipe in place, $1.50 per linear foot. Make same allowances as in Problem 1 for overhead, profit, contingencies and engineering.

3. Estimate the cost of the Oak Grove Ave.-Shattuck St. sewer, the profile of which is given in Fig. 53. Use the unit costs and trench width given in Problem 1, with the following additions:

8-in. vitrified pipe laid, at \$0.75 per linear foot
15-in. vitrified pipe laid, at 1.50 per linear foot
18-in. vitrified pipe laid, at 2.00 per linear foot

Compute excavation to 2 in. below invert grade.

4. Make an estimate of the cost of the combined settling tanks and dosing tank shown in Fig. 223.

Allow 2 ft. excavation on all sides for formwork; excavation, \$1.50 per cubic yard; backfill, \$0.50 per cubic yard; concrete, \$18.00 per cubic yard; forms, \$0.35 per square foot; reinforcing steel, 100 lb. per cubic yard of concrete, \$0.05 per pound; damp-proofing outside of walls and roof, \$0.05 per square foot; manhole frames and covers, \$20.00 each; manhole steps, \$0.60 each. These unit prices include contractor's overhead and profit. Allow 5 per cent for omissions and 15 per cent for contingencies and engineering. (Note that pipes through walls, dosing siphon, etc., are omitted.)

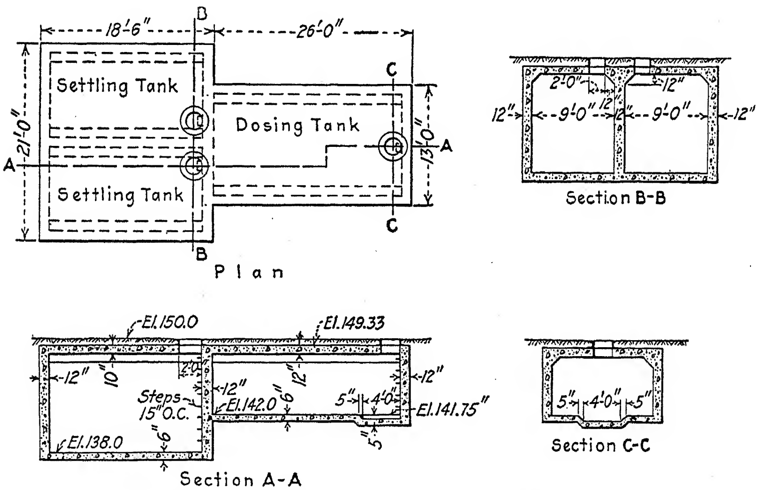

FIG. 223.—Settling tanks and dosing tank for Problem 4.

5. Estimate the cost of two square trickling filters, one 200 by 200 ft. inside of walls with stone 10 ft. deep (Fig. 224), and the other of twice the area with stone 5 ft. deep. Make the following assumptions: original ground is level and at same elevation as stone in filter; excavation 2 ft. beyond neat lines of structure on all sides; thickness of floor, 12 in.; concrete walls 12 in. thick, 12-in. freeboard; 24-in. cast iron distributing pipe and 12-in. laterals; laterals 13 ft. on centers; nozzles 15 ft. on centers along laterals. Unit prices: earth excavation, \$1.25 per cubic yard; concrete, including forms and reinforcing steel, \$35 per cubic yard; filter floor, \$5.00

per square yard; 24-in. cast iron distributor, \$5.50 per linear foot in place; 12-in. cast iron pipe laterals, \$2.50 per linear foot; nozzles, \$5 each; filter stone, \$3.50 per cubic yard. Assume profit included in prices given. Add 5 per cent for omissions and 15 per cent for contingencies and engineering.

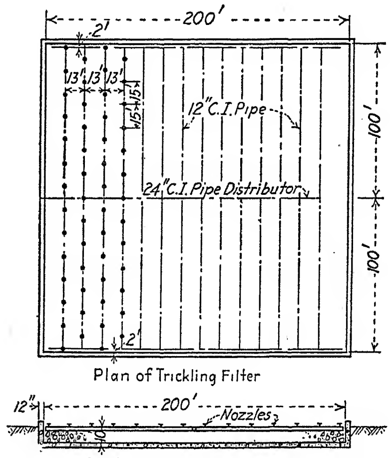

FIG. 224.—Plan and section of trickling filter for Problem 5.

6. With the construction and operation costs of tanks with trickling filters and activated-sludge plants as given in Table 110, compute the cost of construction and the operating cost of each type of plant for an average flow of 10 m.g.d. Assuming that the city borrows money at 4 per cent, that the replacement fund is as given in the table and that costs are capitalized at 4 per cent, determine the capitalized cost of each type.

7. What is the total amount of money paid on a \$10,000 serial bond which is to be paid off in \$1,000 yearly payments with interest paid semi-annually at an annual rate of 5 per cent?

8. What is the present worth of the amounts to be paid in the preceding problem if the value of money to the taxpayers is 6 per cent?

9. Fig. 52, shows the sewerage map for sewers in the Carlisle Brook Drainage District of Springfield, Mass. Fig. 53 is a profile of the main sewer of this district. The estimated cost of the sewers must be paid by special assessment based upon frontage. Figure the rate of assessment per foot of frontage of property served. Exempt corner lots for a depth of 60 ft. on one street. Use cost estimate determined in Problem 3, estimating the cost of all laterals at the cost per linear foot computed for the 8-in. Oak Grove Ave. sewer, except the Burr St. sewer, which costs \$1.00 per linear foot additional.

10. At a sewage treatment plant it is estimated that the screening equipment, valued at \$10,000, has a useful life of 10 years, while the pumps at the

same plant, valued at $15,000, require replacement in 30 years. Compute from Table 111 the fund to be set aside annually at 4 per cent to take care of these replacements.

11. Table 110 gives approximate unit costs of construction and operation of sewage treatment plants. If we assume the costs are as of Jan., 1929, and that the Eng. News-Rec. curve (Fig. 221) applies to this type of construction, what would have been the cost of tanks and trickling filters per million gallons daily if built in Jan., 1916?

ATOMIC WEIGHTS OF CERTAIN CHEMICAL ELEMENTS

Symbol	Name	Atomic weight
Ag	Silver	107.880
Al	Aluminum	26.96
As	Arsenic	74.96
Au	Gold	197.2
Ba	Barium	137.37
Bi	Bismuth	209.00
Br	Bromine	79.916
C	Carbon	12.000
Ca	Calcium	40.07
Cl	Chlorine	35.458
Co	Cobalt	58.97
Cr	Chromium	52.01
Cu	Copper	63.57
Fe	Iron	55.84
H	Hydrogen	1.0077
Hg	Mercury	200.61
I	Iodine	126.932
K	Potassium	39.095
Mg	Magnesium	24.32
Mn	Manganese	54.93
N	Nitrogen	14.008
Na	Sodium	22.997
Ni	Nickel	58.69
O	Oxygen	16.000
P	Phosphorus	31.024
Pb	Lead	207.20
Pt	Platinum	195.23
S	Sulphur	32.065
Sb	Antimony	121.77
Si	Silicon	28.06
Sn	Tin	118.70
Zn	Zinc	65.38

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